CONTAMINANT TRANSPORT FROM LANDFILLS

A thesis submitted in fulfillment of the requirements
for the award of degree of

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in

CIVIL ENGINEERING

By

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CERTIFICATE

This is to certify that the thesis entitled **“Contaminant Transport from Landfills”** submitted by Mr. Bharat Jhamnani for the award of degree of Doctor of Philosophy in Civil Engineering is based on the bonafide research work carried out by him during the period January 2006 to September 2009 under my supervision. Mr. Bharat Jhamnani fulfills the requirements of the regulations laid down for the Ph.D program of University of Delhi, Delhi.

To the best of knowledge, the work presented in this thesis is an original contribution and has not been submitted, in either partial or full, to any other University or Institute for the award of any degree or other similar title or recognition. He is allowed to submit the work for the award of Ph.D in Civil Engineering in University of Delhi, Delhi.

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CANDIDATE'S DECLARATION

I hereby declare that the work presented in this thesis titled **"Contaminant Transport from Landfills"** submitted for the award of degree of Doctor of Philosophy in Civil Engineering, is an authentic record of my own research work carried out under the guidance and supervision of Prof. S.K. Singh, Professor &Head, Department of Civil & Environmental Engineering, Delhi Technological University, Delhi (Formerly Delhi College of Engineering).

The work presented in this thesis is an original contribution and has not been submitted in either partial or full, to any other University or Institute for the award of any degree or other similar title or recognition.

Bharat Jhamnani

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Bharat Jhamnani

ABSTRACT

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ABSTRACT

Point sources such as landfills, can release high concentrations of contaminants into the groundwater because of migration of leachate from its bottom, which is generated primarily as a result of precipitation falling on an active landfill surface, leaching out the potential organic and inorganic contaminants from landfilled waste and discharging the same to groundwater in underlying aquifer. To protect the groundwater from contamination, it is quite essential to provide the bottom barrier of suitable thickness.

The present study was undertaken to determine the rate of movement of potential contaminants from the bottom of a landfill, so as to evolve a rational method for the determination of thickness of landfill liner. A mathematical model was formulated to express the mass transport of contaminants from a landfill due to the migration of leachate. Various mechanisms of contaminant migration from the bottom of landfill were taken into account. The solution of the model in the form of concentration profile of potential contaminants below landfill was obtained using the explicit Finite Difference Method implemented in MatLab 7.0. The model developed was validated for two cases of field data. The simulations of model were run for a range of permeability values, equivalent height of leachate, and retardation factors.

The results of such simulations in the form of maximum relative concentration at various depths, at the end of design period of landfill were used for preparing the design charts. For the conservative contaminant, design charts have been prepared for a range of equivalent height of leachate to represent the mass of landfilled waste.For the non-conservative contaminant, design charts have been prepared for a range of retardation factors. For the purpose of design charts the simulation of the model was run with finite mass of the contaminant as the upper boundary. Initial concentration (background) was assumed to be zero for a fully flushed boundary. The model gives the spatial and temporal variation of concentration along the thickness of the barrier and the maximum concentration recorded in each depth for the entire domain (50 Years). The simulation was run for time domain of 50 years, and the time step was

determined after satisfying the stability criteria of solution. The design charts so prepared can be used for determination of minimum thickness of landfill liner on the basis of maximum permissible relative concentration of contaminant species of interest at the bottom of landfill liner. Design charts were prepared for both conservative and non conservative contaminant.

The contaminant transport model developed in this study was applied for determining the impact of Bhalaswa landfill at Delhi on the groundwater in its vicinity. For determining the impact of landfill on groundwater of the region, simulations of the model were run for tracer contaminant Chloride, and the results of such simulations were compared with the observed values of Chloride concentration in the groundwater of the region. Groundwater samples were collected at varying radial distances from the landfill and in the direction of groundwater flow and analyzed for key contaminants. The results of such sampling and analysis show gradually decreasing concentration of the contaminants away from the landfill. Gas generation from Bhalaswa landfill has been estimated using public domain software based on first order kinetics.

Analysis of the model results was carried out to determine the impact of model parameters viz. time period of simulation, equivalent height of leachate, depth on the transport of contaminants from Bhalaswa landfill. The results of simulation show that Equivalent height of the leachate will have only marginal effect on the transport of chloride or DOC from Bhalaswa Landfill.

The expected gas generation from Bhalaswa Landfill site were computed using public domain simulation software based on first order kinetics. The results of simulations show a considerable potential of gas generation form such landfills which can be collected and utilized for energy recovery.

Experimental studies were carried out on eight admixtures (five prepared using synthesized organoclay and three prepared using manufactured organoclay with bentonite and natural soil as remaining constituents) for the determination of their hydraulic and sorption characteristics.

These admixtures were subjected to laboratory investigations with the objective of investigation the potential use of organoclays with soil-bentonite admixtures in retarding the movement of contaminants while maintaining required permeability specifications. Such amendment is likely to have impact on the minimum thickness required. In order to evaluate the potential effectiveness of sorptive materials as amendments to clay liners, sorption and permeability tests were performed on admixtures obtained by adding varying amount of the organoclay to the natural soil, and purified bentonite. The results of such studies have been used for transport modeling simulations to determine the effectiveness of each of these admixtures with regard to retardation of organic contaminants from landfill leachate and consequent impact on minimum thickness of liner. Design charts were drawn for the determination of minimum liner thickness based on organic contaminant, and using these admixtures. Retardation factor of the admixtures was found to increase with increase of organoclay content. The sorption behavior of all the admixtures have been found to be consistent and scalable.

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Nomenclature

C/N	Carbon to Nitrogen ratio
MSW	Municipal Solid Waste
LFG	Landfill Gas
Btu	British Thermal Unit
v	Advective velocity
n	Porosity
v_a	Darcy Velocity
A_0	Area of Landfill
c	Concentration of Contaminant
D_e	Effective Molecular Diffusion Coefficient
D_h	Hydrodynamic Dispersion Coefficient
D_{md}	Coefficient of mechanical dispersion
α	Dispersivity
R_f	Retardation factor
ρ	Mass density
H_f	Equivalent height of leachate

Chapter 1

Introduction

CHAPTER I

INTRODUCTION

1.1 Introduction

The integrated solid waste management practice advocates a hierarchy comprising of waste reduction at source, recycling and reuse including composting and other techniques for energy recovery and volume reduction, followed by landfilling. The materials which can't be recovered finally end up in landfill for ultimate disposal. Sanitary landfilling is the preferred method of Municipal Solid Waste (MSW) disposal due to its favorable economics.

With the ever increasing awareness about the problems related to environmental issues, landfills have captured the attention of many researchers. The impact of landfills, relatively unknown so far, has come under an ever increasing scrutiny by the people/agencies. The new developments in the area of environmental geo-technology and the strict regulations for design of landfill facilities, construction, operation and closure have made the landfills less environmentally offensive. The stringer requirements for an engineered sanitary landfill design have resulted into this final disposal of solid waste into a costly affair.

One finds the research and development activities in solid waste management to be at low priority in India or for that matter in most of the Asian countries. Such a situation often leads to selection of inappropriate technology in terms of local climatic conditions, financial and human resource capabilities and social or cultural acceptability.

Like most of the Asian countries, landfilling method followed in India is not in keeping with the modern practices of engineered sanitary landfill and the wastes are largely dumped at a site. This dumping is normally carried out in low lying areas that are prone to flooding and are often vulnerable to contamination of groundwater. Groundwater pollution which is not assessed is another threat posed by dumping of solid wastes.

1.2 Leachate: Characteristics and its Impact on Groundwater

Leachate is generated primarily as a result of precipitation falling on an active landfill surface although other contributors to leachate generation include groundwater inflow, surface water runoff, moisture from emplaced waste, and biological decomposition.

Leachate may contain dissolved or suspended material associated with wastes disposed off in the landfill, as well as many byproducts of chemical and biological reactions. Leachate from municipal solid waste landfills varies in strength as a result of the biological activity occurring as the solid waste decomposes. Leachate from young landfills has both high dissolved solids content as well as a high concentration of organic matter as compared to domestic wastewater. Leachate may also contain trace amounts of hazardous constituents found in the waste stream.

Gravity causes leachate to move through the landfill to the bottom and sides and through the underlying soil until it reaches the groundwater zone or aquifer. Leachate tends to migrate from the landfilled waste resulting in contamination of the underlying soil and ground water.

As leachate moves down the subsurface, it mixes with groundwater held in the soil spaces and this mixture moves along the groundwater’s flow path as a plume of contaminated groundwater.

1.3 Significance of Groundwater Contamination

As per an estimate of the Central Pollution Control Board (CPCB), the landfills of the National Capital Territory (NCT), Delhi, cumulatively generate a significant amount of leachates annually, which is alarming in terms of its groundwater contamination potential (CPCB 2001). In many parts of India, especially in the arid and semiarid regions, due to the vagaries of monsoons and scarcity of surface water, dependence on the groundwater resource has increased tremendously in recent years. Viewed from the international standard that ''<1,700 m^3/person/year'' qualifies as water-stressed, India is water-stressed today and is likely to face severe water scarcity by 2050 (CPCB 2001). Delhi, as the rapidly growing capital city of Asia, is facing problems in terms of both the groundwater quality and quantity.

Any contamination of this scarce groundwater resource in the country would be a significant strain on the water distributing unit because rehabilitation of polluted groundwater is expensive, and often may not be technically feasible. Secondly, this means that water has to be abstracted from other sources and this would result in conflicts with other uses, especially if the land is facing a drought period, which is often the case. The severity of this water resource acquires significance on account of its vulnerability to pollution, the associated complexity in pollution source identification, limited feasible options for treatment of groundwater, insufficient dilution and lack of any natural cleansing capability. As a sizeable population in India is dependent on groundwater as the only source particularly densely populated areas where such contamination is more commonly noticed, it is necessary to pay urgent attention to all the activities contributing towards declining quality and quantity of the groundwater resource, landfilling being one of them.

1.4 Controlled Landfill Design

A sanitary landfill is defined as a system in which municipal solid wastes are disposed off, compacted, and covered with a layer of soil at the end of each day's operation. The planning, analysis, and design of modern land disposal systems involve the application of a variety of scientific, engineering, and economic principles. Engineering principles are followed to confine the wastes to the smallest possible area, to reduce them to the smallest particle volume, and to cover them after each day's operation to reduce exposure to vermin (Tchobanoglous et al. 1997). The essential components of a MSW landfill are a liner system at the base and sides of the landfill, a leachate collection and control facility, a gas collection and control facility, a final cover system at the top, a surface water drainage system, an environmental monitoring and a closure and post-closure plan for long-term monitoring, operation and maintenance of the completed landfill.

The concepts of containment systems for modern sanitary landfills involve the use of barrier layers at the bottom of landfill to prevent leachate from leaving the landfill and contaminating the underlying soil and groundwater, and to prevent water from entering the landfill to create leachate. Barrier layers are constructed of materials that possess a low permeability to water. The most common materials include compacted soil (clay). Natural clay deposits are sometimes used as landfill barrier layers. In most sanitary landfills however, clay liners are constructed by modifying the structure of the clay soil brought to the site by the addition of water and mechanical compaction to achieve optimum engineering characteristics. A number of properties make compacted soil amenable to use as a component in a landfill containment system. These include mechanical properties such as shear strength but most importantly the permeability of the clay to landfill leachate. Most engineered clay liners must meet requirements for hydraulic conductivity of less than 10^{-7} cm per second.

1.5 Motivation for Present Study

To comply with the pollution prevention criteria of the region the landfills need to be provided with the bottom barrier, thickness of which needs to be determined based on sound engineering principles which involves modeling of contaminant migration processes from landfill into the clayey barrier. The problem of migration of contaminants through natural barriers located below landfills has been addressed by many researchers in the past, but most of the methods developed so far have been analytical or semi analytical and each one worked under a set of limitations associated with their respective models. These models have limited application however some of these models are popular because they are simple and user friendly. The softwares available in public domain for the modeling of contaminant transport are not suitable for application to the problem of contaminant transport from landfills because of the special boundary conditions required. Moreover many of the softwares related to contaminant transport in porous media are neither available in public domain nor through any agency in India. There exists an absence of information and knowledge to face the challenges posed by solid disposal problem in Indian conditions with regard to barrier design where one finds a large variation in the quantity of wastes to be handled in varying hydrogeological conditions. There was thus, this gap which led to taking up this study for the development and validation of the mathematical model for transport of contaminants from the bottom of landfill, and consequent preparation of design charts on the basis of results of model simulations which can be readily used for the determination of minimum barrier thickness.

Further the modifications in the properties of landfill liner materials if needed can be achieved by adding certain special materials depending upon the function desired. Such materials are added to the landfill liner for getting enhanced retardation of target contaminants. Different materials may be added depending upon the type of contaminant and to the extent of desired retardation. Clays modified with cationic surfactant (organoclays) have been known to be having a considerable potential with regard to sorption and consequent retardation of organic compounds form landfill leachate. A study carried out by Lo (Lo 2003), have shown that the retardation factor of the landfill barrier against organic contaminants can be obtained as high as about 12.5 by amending such materials with 20% of organoclay. The enhancement of retardation factor will have direct bearing on the minimum thickness of landfill barrier required to comply with pollution prevention criteria in a region. A higher retardation factor shall lead to greater reduction in the thickness of the barrier required on the basis organic contaminants. No such studies have been undertaken in India so far and import of such amendments if required can run into huge costly affair, unthinkable for municipalities in India. It was thus thought of to take up a study involving the use of liner materials amended with locally available organoclay, for the determination of the impact of such amendments on the sorption and consequent retardation of organic contaminants in landfill leachate, using the leachate obtained from Bhalaswa landfill site in Delhi. The study was thus aimed at determining the impact of amending the barrier materials on retardation of organic contaminants from landfill leachate using locally available materials, and under Indian conditions.

1.6 Objectives of Study

Looking to the current practice of municipal solid waste disposal practice being followed in India and the extent of likely impact on groundwater contamination, a study was taken with the focus on identification of groundwater contamination potential of landfill and control measures for the prevention of such contamination. Following were the objectives of present study:

1. To develop and validate mathematical model that would address the problem of migration of contaminants from the bottom of landfill.
2. To apply the model developed in this study for the determination of likely impact of Bhalaswa landfill site at Delhi on the contamination of groundwater in its vicinity.
3. To prepare the design charts which may be used for the determination of minimum thickness of landfill liner for a typical hydrogeological conditions and pollution prevention criteria.
4. To study the various admixtures for landfill liners for the control of migration of contaminants from landfill leachate.

1.7 Scope of the Study

Scope of present study was limited to the development and validation of mathematic model to address the problem of contaminant migration from the bottom of landfill, taking into account the various mechanisms of contaminant migration in such a case. The case study of Bhalaswa landfill site was limited in the scope to the application of model to the landfill site for the purpose of model validation and determination of likely impact of landfill on the groundwater in the vicinity of landfill site. Preparation of design charts for the landfill liner thickness was limited to plotting of maximum relative concentration of key contaminants below landfill with respect to depth on the basis of simulation results of model. Studies on admixtures were limited to the determination of the model parameters for admixtures prepared in the present study and consequent evaluation of their effectiveness with regard to contaminant retardation.

Chapter 2

Literature Review

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The landfill as we know it has evolved from a long tradition of land disposal of waste. Land disposal of waste often as dumps was subject to aesthetic, safety, and health problems that prompted innovations in design and operation. Production of leachate has led to documented evidence of groundwater and surface water contamination. Landfill gas emissions can lead to malodorous circumstances, adverse health effects, explosive conditions, and global warming. Traffic, dust, insect vectors of the diseases and noise from a landfill site often are objectionable to nearby neighbours. These issues have lead to strict regulation of the landfill disposal method. The sanitary landfill evolved in response to indiscriminate dumping of solid wastes. Sanitary landfilling is the preferred method of Municipal Solid Waste (MSW) disposal due to its favorable economics. Effective pollution control through the proper design of landfill and leachate management facilities requires an understanding of leachate quality and quantity. The rate and characteristics of leachate and landfill gas produced from a landfill vary from one phase to another and reflect the microbially mediated processes taking place inside the landfill (Augenstein and Pacey 1992). A review of the literature highlights the research efforts directed towards various aspects of solid waste disposal by landfilling viz. quantity and quality of leachate, leachate migration from landfills, estimation of landfill gas emissions, landfill liner amendments. Each of these is discussed in detail as follows:

2.2 Leachate Quantity

Determination of expected quantity of leachate generated is crucial to the design of Leachate Collection and Removal (LCRS) system. The total quantity of leachate generated at a given waste containment system is primarily a function of the quantity of water infiltrated into the system and quantity of fluids generated within the waste. The former is turn in dependent on a number and intensity of climatologic and hydrologic processes, primarily rainfall, runoff and evaporation. Thus, to estimate the quantity of leachate, one needs to conduct a water balance for the entire system. Leachate quantity is usually modeled and/or determined using a simple water balance approach taking into account the amounts of water entering the landfill (i.e. precipitation, waste moisture in excess of moisture holding capacity of the waste and additional water input such as water in wastewater treatment plant sludge if allowed) and the amounts of water leaving the landfill (i.e. water consumed in biochemical reactions and evaporation). The water balance method assumes that precipitation either runs off from the landfill or infiltrates the surface (uncovered refuse, intermediate cover, or final cover). Some of the infiltrated water evaporates from the surface and (or) transpires through the vegetative cover if it exists. Some of infiltration may make up a deficiency in soil moisture storage, the difference between field capacity (FC) and the existing moisture content (MC). The remainder of infiltration moves downward forming percolate (PERC) and eventually leachate (L) as it reaches the base of the landfill. PERC may be augmented by infiltration of groundwater (Fenn et al. 1975).

The water balance methods are based on procedures developed by Thornthwaite (Thornthwaite and Mather 1955, 1957; Thornthwaite 1964) in the soil and water conservation field. Since his work many research efforts have been directed to develop the water balance equations in the last 50 years (Fenn et al. 1975; Perrier and Gibson 1980; Knisel and Nicks 1980; Skaggs 1980; Schroeder et al. 1984a, 1984b). These water balance models consider the landfill a "black box," requiring only a material balance of water flow into and out of the system. The basic water balance equation used to develop the model is:

$$L=P-ET-R-\Delta S \qquad \text{.....2.1}$$

where:

L= the leakage volume produced
P= precipitation falling on the surface
ET= water lost due to evapotranspiration
R= water lost due to runoff
ΔS= the change in moisture storage volume

The water balance models have been used extensively in predicting leachate quantity and aiding design of landfills. Water balance model predictions may be suspect due to the questionable accuracy of the input parameters, such as, rainfall, evapotranspiration, permeability, and refuse moisture storage estimates (Bagchi 1994; El-Fadel et al. 1997). The second approach to predicting landfill flow is using finite-difference/finite-element solution techniques. Many investigators have taken this more complex approach of using the unsaturated flow theory through porous media to predict landfill leachate flow (Korfiatis 1984; Straub and Lynch 1982a, 1982b). The Hydrologic Evaluation of Landfill Performance (HELP) is the most widely used model which employs the same concept of water balance excluding the biochemical reactions and computes the leachate quantity based on a detailed meteorological and waste characteristics data.

2.3 Leachate Quality

The chemical composition of leachate depends on many factors such as the initial composition of solid wastes, particle size, degree of compaction, hydrology of the site and the age of the tip, the last one reflecting the degree of stabilization of the wastes. The quality of leachate is further affected by several pretreatment methods or management practices such as the shredding of wastes, the separation of recoverable material (i.e. paper, aluminum, and glass), the recycling of leachate back to the landfill, or the co-disposal of municipal wastewater sludge. For these reasons great variations in the quality of leachates produced from different landfills have been reported in the literature (Johansen and Carlson 1976; Chian 1977; Bookter and Ham 1982; Harmsen 1983; Kouzeli-Katsiri et al. 1993; Robinson 1993; Gettinby et al. 1996; Farquhar1989). The estimation of the volume of leachate and the concentration profiles of the main contaminants present is of particular importance for the proper design and operation of sanitary landfills. This knowledge permits the planning of facilities required for the collection and treatment of leachate, including the recirculation of leachate back to the landfill. Moreover it would provide an estimate as to the duration of the process of decomposition and the possible life span of the landfill.

The concentration profiles of several leachate constituents measured at the bottom of laboratory-scale columns filled with compacted municipal refuse have been reported in the literature by several authors (Pohland 1975; Fungaroli and Steiner 1979; Raveh and Avnimelech 1979; Ham and Booker 1982; Ehrig 1988, 1991; Colin 1990). Although wide differences in leachate composition were noted, in general the concentration curves resemble a hydrograph with a rising limb, a peak, and a recession limb.

Attempts to model and predict municipal solid waste leachate composition and volume have generally followed two approaches. The first and simplest approach has been to fit empirical equations to contaminant concentration curves (Raveh and Avnimeleh 1979; Wing 1979; Reitzel et al. 1992). The contaminant curve is generally developed as either contamination concentration versus time or cumulative leachate volume per unit mass of refuse. Although these models capture the general decrease from high initial concentrations to low final concentrations, they tend to be site-specific, to the particular landfill or lysimeter.

In the second approach, models were developed that quantitatively describe the biological processes occurring during leaching. The model used almost exclusively by several authors was developed originally by Straub and Lynch (Straub and Lynch 1982a). These authors used four process equations to describe the solubilization of organic matter, the degradation of soluble organic matter and the growth of acidogenic and methanogenic anaerobic biomass. A pseudo first-order kinetic was used for the solubilization rate whereas the Monod model (Monod 1949) was used for the degradation rate and microbial growth rates. The lysimeter was simulated as a series of fully mixed reactors. They used data from the literature to calibrate the models and derived values for the kinetic coefficients. A satisfactory simulation was obtained when the reactor was depicted with four fully mixed compartments in series. In a second paper, Straub and Lynch (Straub and Lynch 1982b) used the theory of unsaturated flow through porous media to describe the flow through the lysimeter. The flow equations were solved explicitly. Using the same kinetic parameters calculated previously, these authors found no significant differences between the two models. They concluded that the fully mixed reactor assumption offered a good simplification for the simulation of the decomposition processes in lysimeter. The same unsaturated flow model was used by Korfiatis and Demetracopoulos (1984) and Demetracopoulos et al. (1986). These authors employed a different numerical scheme to solve the flow equations and performed a sensitivity analysis for the evaluation of equation parameters. Their results were not, however, verified with the experimental data. The simple four-reactor model developed by Straub and Lynch (1982a) was subsequently used by Colin

(1990) to predict the leachate quality from laboratory lysimeters. Gonullu (1994a, 1994b) followed the same model but with a single fully mixed reactor to predict the quality of leachate from two laboratory-scale lysimeters. Using multiple correlation analysis of the experimental data, he developed two equations to predict the maximum concentration of substrate in the effluent and the value of the microbial kinetic coefficient.

Kouzeli-Katsiri et al. (1993, 1999) developed a simple mathematical model to simulate solid waste decomposition in landfills. They took into account the processes of exchange of the organic matter between the solid and the liquid phase and the depletion of the dissolved organic matter due to biological decomposition and flushing. They employed two equations using first-order kinetics to describe these processes. In their study the landfill was considered as a single fully mixed reactor. Their model has a smaller set of parameters than other models proposed in the literature. They calibrated their model using the results obtained from six pilot laboratory-scale lysimeters. They proposed kinetic parameters for characteristic types of solid wastes and landfill management practices. The experimental setup used by them consisted of six lysimeters made of PVC with a height of 2.5 m and internal diameter of 0.30 m which were insulated externally with a 3.5 cm thickness of polyurethane foam. Fresh refuse used by them was obtained from the Ano Liossia tip, which receives solid wastes from the major Athens area with a population of 3,200,000. The waste placed in each lysimeter was compacted to a 1:3 ratio and was covered by a final layer of 30 cm sand. Simulated rainfall, corresponding to the mean monthly rainfall at the Ano Liossia tip was added at the top of the lysimeter at weekly increments. Lysimeters 1 and 3 were filled with fresh refuse, and lysimeters 2 and 4 with shredded refuse. Lysimeter 5 was half-filled with fresh refuse and half-filled with already decomposed refuse taken from a 2 year-old deposit at the Ano Liossia tip. Finally, lysimeter 6 was filled with a mixture of fresh refuse and anaerobically digested dewatered sludge in a ratio of two parts refuse to one part sludge. Leachate was subjected to subsequent analysis. Leachate quality was measured at monthly intervals in cumulative samples collected over this period. Leachate volume was monitored at weekly intervals. They concluded that due to the very slow rate of reactions, the landfill can be considered as a single fully mixed reactor with a volume equal to the volume of the water content of the deposited mass of refuse at field capacity. They described complex processes occurring during refuse decomposition by two first-order reaction equations for organic matter solubilization and soluble organic matter depletion. The total leachable mass of COD was expressed as a fixed percentage of the total mass of wastes. Their model thus contained three kinetic parameters, the values of which were obtained from the experimental results. They further concluded that the single most important factor affecting the rate of decomposition is moisture. Thus, an increase in moisture, either by rainfall addition or leachate recirculation, could increase the rate of biodegradation. They found that the shredding of the wastes does not have any significant effect either on the concentration of COD in leachate or on the time of refuse decomposition. The addition of municipal wastewater sludge to the refuse caused an almost 10 fold increase in the rate of decomposition.

2.4 Migration of Landfill Leachate

Gravity causes leachate to move through the landfill, to the bottom and sides, and through the underlying soil until it reaches the groundwater zone or aquifer. Leachate tends to migrate from the landfilled waste, resulting in contamination of the underlying soil and ground water. As leachate moves down the subsurface, it mixes with groundwater held in the soil spaces and this mixture moves along the groundwater’s flow path as a plume of contaminated groundwater. The impact of landfill leachate on the contamination of groundwater depends upon various parameters viz. properties of flow and media through which flow of leachate takes place, and the properties of contaminant, density of waste etc. An exercise to determine the likely impact of leachate migration on groundwater contamination involves taking into account the parameters and processes and involved and formulating all those in terms of mathematical model (Kelley 1976).

Review of literature highlights the contributions made by researchers to determine the model parameters like diffusion coefficients in a saturated and unsaturated natural barrier, hydraulic conductivity of barriers and adsorption characteristics for heavy metal removal. The design of an earthen barrier system in the case of buried waste traditionally is based on the assumption that the hydraulic conductivity controls the rate of leachate migration. However, recent field studies have indicated that the diffusion is the controlling mechanism of contaminant transport in many fine grained soil (Bagchi 1994)

Rowe (1988) discussed in detail the mechanisms that control the migration of contaminants and the method of modeling the same. He also discussed ways to determine diffusion and distribution coefficients and introduced the concept of equivalent leachate height. These concepts have been used in formulating the model used in this study.

Conca and Wright (1990) determined diffusion coefficients of unsaturated gravel and showed that the quartzite gravel had the lowest coefficients. Likewise Rowe and Badv (1996a, 1996b) determined the dispersion and diffusion coefficients of chloride in clayey silt, silt and sand for Darcy velocity of the order of 0.018 m/yr. In their study, the effective diffusion coefficient in saturated fine gravel was found to be about 93% greater than that for unsaturated fine gravel. Tests conducted in low velocities encountered in a typical landfill site and the modeling results show that there is no significant mechanical dispersion evident in clay, silt or sand under such low flow rate.

Shakelford and Daniel (1991) determined the factors that affect the diffusion in aqueous or free solutions and suggested that diffusive transport is slower in free solution as compared to soils. Later, Shakelford and Redmond (1995) also obtained the hydrodynamic dispersion coefficient in fine grained barrier material and concluded that diffusion dominates miscible transport at low flow rates in such barriers. Rowe et al. (1988) obtained diffusion and distribution coefficients in saturated undisturbed clayey soils and validated their theoretical model using data obtained from an experimental study. They determined the diffusion and distribution coefficients of a contaminant using undisturbed clayey soil samples while maintaining a Darcy velocity in the range of 0.025 to 0.035 m/yr. This range exceeds the normal velocity in most practical applications involving clayey liners. According to them, the contribution of mechanical dispersion to the hydrodynamic dispersion coefficient at a velocity of 0.035 m/yr or less is negligible for clay and diffusion is the dominant mechanism for contaminant transport through clay for seepage velocity between 0.064 m/yr to 0.09 m/yr.

Rowe and Booker (1995) analyzed one dimensional contaminant transport model for single solute in a layer of finite thickness. In their study, they considered the combination of the effects of advection, diffusion, dispersion and chemical retardation due to finite quantity of pollutant in the landfill overlying a clay deposit with moving groundwater beneath the clay deposit. Their model also included leachate mounding and the effect of clogging of leachate collection system due to the contaminant. However, their analytic solution was suitable only for cases where homogenous liner thickness was small compared to the dimensions of landfill.

Rowe and Booker (1995) applied semi-analytic technique to a barrier system consisting of a primary leachate collection system, a primary clay liner, a secondary leachate collection system and secondary liner system overlying a fractured aquitard located above an aquifer. This new technique accommodates the varying properties of the system, complex source concentration history and movement of contaminant through a fractured aquitard. The time and cost involved while performing the analysis is also relatively less, however it’s application is limited to a horizontally layered soil with the soil properties being same at ant horizontal location within the layer.

Similarly some researchers have developed the models (Benson and Daniel 1994a) that predict the minimum thickness of the soil liners using a stochastic approach but they only considered advective flow in saturated soil. Further Benson and Daniel (1994b) discuss some difficulties encountered in the analysis of soil liners. They suggested that the performance of soil liners depend on the minimum travel time and the magnitude of flux subsequent to the first passage measured using probability theory to incorporate the spatial variations of the hydraulic conductivity properties. They recommended minimum thickness of compacted liners should be 0.6m to 0.9m with respect to spatial variability of hydraulic conductivity.

Jessberger (1997) discussed controlled landfill design and highlighted the various aspects of design followed in European countries. Rowe and Nadarajah (1993, 1997) presented analytical methods of predicting the velocity of low beneath the landfill and ways of evaluating the hydraulic conductivity of aquitard.

Munro et al. (1997) investigated the retardation of contaminants in shallow clayey soils and later validated their field studies with analytical models of Sudicky (1988). Sawbrick (1994) gave a comprehensive list of all the processes that take place in waste deposits.

Boateng and Cawlfield (1999) conducted a two dimensional sensitivity analysis of contaminant transport in the saturated zone and suggested that the bulk density and distribution coefficients are significant for reactive transport.

An analytical model to assess the risk of groundwater pollution caused by leaks from solid waste depositories was developed by Rudakov and Rudakov (1999). These analytical modeling approaches for design of clayey liners are simple to use, but overly conservative, hence one needs numerical solution.

Yan et al. (2006) presented a contaminant transport model based on fuzzy set theory to simulate water flow and contaminant transport in the unsaturated soil zone under surface ponding condition. They found that saturated hydraulic conductivity is the most sensitive to model outputs among all the parameters. They used trapezoidal fuzzy numbers to express the uncertainties associated with saturated hydraulic conductivity.

Tsau and Aarne (1998) discussed improved landfill liner for the prevention of groundwater contamination due to heavy metals. They found that the hydraulic conductivity of clay lime, first increases with increasing lime and thereafter decreases with increasing lime. They concluded that the incorporation of lime in landfill liners reduces hydraulic conductivity, increases puncture resistance, and enhances heavy metal capture capacity.

Tchobanoglous et al. (1997) studied the moisture content of solid wastes, and concluded that for most MSW the moisture content varies from 15 to 40 %, depending on humidity and weather conditions, waste composition, and the season of year.

Michaels and Lin (1954) studied the factors affecting the hydraulic conductivity of low permeability liner materials and concluded that the nature of permeant can significantly affect the hydraulic conductivity of soil. The large variations in permeability at a given void ratio can be explained by the differences in soil structure resulting from using different fluids during sample preparation. Hydraulic conductivity varies with the cube of degree of saturation, which means that the permeability should increase as the degree of saturation increases.

Mitchell and Madsen (1987) reviewed literature on test data related to chemical effects on clay hydraulic conductivity and found that in general inorganic chemicals may influence hydraulic conductivity of clays through their effects on soil fabric. The results indicated that the reduced electrolyte concentration led to a substantial decrease in hydraulic conductivity because of dispersion. The effects of sodium hydroxide at pH 9, 10, and 11 showed little effect on hydraulic conductivity. However, hydraulic conductivity decreased by a factor of 10 for pH 13. The conclusions drawn by them related to the effects of chemicals on hydraulic conductivity of clays were that hydrocarbons to concentrations at or below their solubility limit do not affect hydraulic conductivity, permeation with pure hydrocarbons may, however influence hydraulic conductivity, and specific tests may be performed to quantify the change in hydraulic conductivity, alcohol concentrations less than 80 % have little effect on hydraulic conductivity, acetone solutions at concentrations less than 75% have little effect on *k*. However, pure acetone can increase hydraulic conductivity.

Benson and Daniel (1990) suggested that to achieve low hydraulic conductivity, it is necessary to destroy clods either by wetting the soil at high moisture content or using large compacting effort. Gracia-Bengochea et al. (1979) concluded that hydraulic conductivity is controlled by changes in macropore distribution rather than porosity. Mitchell et al. (1965) studied the effects of moisture content on hydraulic conductivity and concluded that higher moisture content result in dispersed structure, resulting in lower value of *k*.

Thus to summarize, a considerable effort have been directed towards determination of transport parameters and evolving methods to solve the problem of migration of contaminants through natural barriers located below landfills. Most of the methods developed so far have been analytical or semi analytical and each one worked under a set of limitations associated with their respective models. These models have limited application, however some of these models are popular because they are simple and user friendly.

Also municipal agencies in India are not well equipped with the knowledge and information required for the sustainable disposal of solid wastes. The agencies are called upon address the problem of varying densities of solid wastes to be disposed off in varying hydrogeologic conditions. There is crunch for the availability of suitable land for the disposal of solid wastes in almost every nook and corner of the country. Thus it was felt, that it is necessary to have rational method for the determining the thickness of landfill liners which would be able to cope up with the challenges posed by varying densities of wastes, and varying hydrogeologic conditions peculiarly observed in Indian conditions. The present study, was therefore taken for the modeling of contaminant transport from landfill, and determine the temporal and spatial distribution of key contaminants from landfill.

2.5 Gas Production from Landfills

The disposal of wastes in landfills results in simultaneous physical, chemical, and biological reactions to take place and allow for waste stabilization and landfill gas generation. Landfill gas is the product of a series of complex reactions involved in the decomposition of organic matter that produce methane and other gases and compounds (Bookter and Ham 1982). Methane is a valuable energy resource and has many applications as such. It is also a potentially dangerous gas that must be controlled and regulated. The major reasons to monitor emissions and/or install LFG collection systems include odor abatement, gas migration inhibition, compliance with government regulations, and to offset costs or make a profit through the sale of fuel or energy.

Methane and LFG yields from waste disposed in landfills are the subject of much research. Mathematical and computer models have been developed to predict gas yields. These models are based on population, per capita waste generation, waste composition and moisture content, percent waste actually landfilled, and expected methane or LFG yield per unit dry weight of biodegradable waste. Mathematical models can also be used to model extraction systems including layout, equipment selection, and operation optimization, failure simulation, and problem determination and location within existing extraction systems (Young 1989). In theory, the biological decomposition of 1 ton of MSW produces 15,574 ft^3 of LFG containing 55 percent methane and a lower calorific value of 530 Btu/ft. Since only part of the waste converts to methane, not all of which is extracted, the actual average methane yield is closer to 3,893 ft /ton MSW. Gas generation may then be more closely estimated by the use of models, and/or by laboratory tests of waste samples, small and large-scale pilot tests, and test cells incorporated in the landfill itself. Sources of error include inability to select representative landfill samples or well sites, inability to accurately determine the volume from which a well is actually extracting, and variation in gas generation across the landfill and over time (seasonally). Of course, once a recovery system has been installed and is functioning at steady-state, reasonably precise recovery data can be collected to predict future results (Augenstein and Pacey 1992).

Pohland (1975) reported on a study of the impact of the moisture content of MSW on gas production rate. They reported that gas production essentially ceased when the percent moisture in the waste is less than about 20%. The rate of gas production increased with moisture content up to the maximum moisture content evaluated, about 60%. It is possible, although not investigated by them, that higher rates of gas production could have occurred with higher moisture content.

Leachate recycle has been used for many years as a means of disposing of MSW landfill leachate and to enhance stabilization of fermentable organics in MSW (Pohland and Harper 1985, Reinhart et al. 1982). The rate of methane generation is controlled by the amount of moisture present in the waste. In the classical sanitary landfill where no attempt is made to restrict entrance of moisture, landfill gas formation typically takes place for 30 to 50 years. As additional moisture is added to the waste, the rate of methane formation increases. It has been well-documented in the literature that by adding moisture through leachate recycle the period during which methane is generated under ideal conditions in a sanitary landfill can be reduced to 5 to 10 years.

2.6 Landfill Liner Amendments

Liner amendments are incorporated into landfill liner materials to impart special properties to it, for the purpose of retarding the migration of specific contaminants from landfill leachate. In this study liner amendments have been tried for the retardation of organic contaminants from landfill leachate. These amendments are supposed to sorb the organic contaminants and thus impede the movements of such contaminants to the bottom of landfill liner.

Shanon et al. (2005) studied the sorptive capacities of granular activated carbon (GAC), shale, benzyltriethylammonium-bentonite (BTEA-bentonite), and hexadecyltrimethylammonium-bentonite (HDTMA-bentonite) for organic compounds. They carried out the experimental studies for sorption of organic compounds by these four materials to evaluate their effectiveness for use in compacted clay landfill liners. They carried out laboratory sorption and permeability experiments on these materials with 0.002 N $CaSO_4$ solution (to simulate the ionic strength of ground water) and a synthetic leachate. From the results of their experimental investigations with 0.002 N CaSO4, they concluded that indicate that all four amendments have a very high sorptive capacity for the three organic solutes viz. benzene, trichloroethylene, and 1,2-dichlorobenzene. They found that GAC exhibited the highest sorptive capacity for all three solutes, followed by BTEA-bentonite, HDTMA-bentonite, and shale. From the results of experimental investigations conducted with synthetic leachate, it was concluded by them that GAC and BTEA-bentonite did not exhibit competitive sorption effects with the synthetic leachate constituents, while shale did exhibit some competitive sorption effects between benzene and trichloroethylene and the synthetic leachate constituents. They carried out permeability tests on specimens composed of Ottawa sand, untreated bentonite, and either 3 or 9% amendment by weight. Their Results indicated that all amended specimens had a hydraulic conductivity less than or equal to 1x 10^{-7} cm/ s, with the exception of the specimen amended with 3% GAC, which had a measured hydraulic conductivity value of 2x10^{-7} cm/s. They found that the changing of permeant fluid to a synthetic leachate had little effect on the overall hydraulic conductivity of the specimens. They further carried out one-dimensional benzene transport simulations for the complete evaluation of these materials for their incorporation into a compacted clay liner. They found that all four amendments can effectively retard the transport of benzene through the liner. They concluded that amending liners with 3 or 9% GAC or 3 or 9% BTEA-bentonite effectively eliminated benzene flux through the liner over a 100-year period. They further concluded that liners amended with HDTMA-bentonite or shale also reduced the benzene flux exiting the liner over the simulation period, but to a lesser extent.

Tarek et al. (2002) conducted field studies to determine if the field hydraulic conductivity of barrier layers constructed with foundry green sand is comparable to the hydraulic conductivity measured in the laboratory on laboratory-compacted specimens. They constructed three test pads with foundry green sand. Their field hydraulic conductivity was measured using sealed double ring infiltrometers, two-stage borehole permeameters, and on large block specimens. Additional field hydraulic conductivity tests were conducted on the test pads after exposure to winter weather causing freeze-thaw cycling and summer weather causing desiccation. They found that the field hydraulic conductivity data followed the same trends with bentonite content and liquid limit observed in the laboratory. Testing after winter exposure showed that the field hydraulic conductivity was unaffected by winter weather, even though the test pads underwent up to six freeze-thaw cycles. They concluded that, exposing the test pads to summer weather had no measurable effect on the field hydraulic conductivity. The results of their field study validated that foundry sand as a useful industrial by-product that can be beneficially used as a hydraulic barrier material.

Lo (2003) studied the feasibility of using modified clays in conjunction with conventional soil-bentonite admixtures as a waste containment barrier with the objective of determining the abilities of such admixtures to act as a sealant and retain dissolved organic contaminants. They prepared five kinds of soil admixture for the batch sorption, compaction, and hydraulic conductivity tests. The first three kinds of soil admixtures consisted of 80% natural soil with 20% organoclay; 10% bentonite and 10% organoclay; 20% bentonite. The fourth soil admixtures consisted of 70% natural soil, 20% bentonite, and 10% organoclay. The last admixture was composed of 60% CDV, 20% bentonite, and 20% organoclay. The various percentages of each soil type were determined on a dry weight basis. The natural soil used in their study was completely decomposed volcanic rock (CDV), yellowish brown in color. Modified clay used in their study was obtained as manufacture product from Biomin International, USA. They carried out experimental investigations on these admixtures for the determination of sorption capacities and hydraulic conductivity. The sorption isotherms of total organic carbon, by five types of soil admixtures were nonlinear. They concluded that the soil specimen with more modified clays exhibits higher organic sorption capacity and a larger retardation factor. The specimens with 20% of pure bentonite by dry weight have a higher optimum water content and plasticity. They found that with the addition of bentonite in the soil material consisting of natural soils and modified clays, the hydraulic conductivity to leachate decreases from about 1 x10-^{7} to 1x 10^{-8} cm/s. No incompatibility between landfill leachate and soil specimens was found. They applied curve fitting technique to determine the effective porosity and hydrodynamic dispersions of both

Chloride and Total Organic Carbon. They carried out 1D transport model simulations to develop design charts for the determination of liner thickness using such admixtures. They concluded that that the presence of modified clays in the soil containing 20% bentonite largely reduces the required liner thickness.

Loretta and Franky (2001) used Bentonite, forest soil, and spruce bark and subjected these materials to batch adsorption testing, leaching cell testing, and selective sequential extractions (SSEs) to investigate the heavy-metal compatibility of clay barriers and the potential of forest soil and spruce bark as clay barrier materials. They concluded that the materials can be ranked on the basis of sorption capacity as forest soil > bentonite = spruce bark. The hydraulic conductivity values of heavy metal leachate were two orders of magnitude greater than those of the blank (0.01 mol calcium nitrate) leachate. The forest soil admixture ranked first in terms of heavy-metal retention capacity and breakthrough points. The mobility of Cadmium was 4.5 times higher than that of Lead, and Copper was 2.5 times more mobile than Lead. The leaching cell and SSE results indicated that the heavy metals cause significant preferential channeling. The SSE results showed that the addition of forest soil and spruce bark to clay barrier mixes promotes heavy-metal fixation. They used three admixture in their study (i) 100:8 ratio (by weight) of sand/bentonite (bentonite admix), (ii) 100:7:1 ratio (by weight) of sand/bentonite/forest soil (forest soil admix), and (iii) 100:7:1 ratio (by weight) of sand/bentonite/spruce bark (spruce bark admix).

Jaruwong and Wibulswas (2003) studied the effect of Quaternary Ammonium Cations (QACs) carbon chain length used for modifying Montmorillonite clay to examine its adsorption properties towards humic acid utilizing a batch equilibration technique. They performed a comparative using Montmorillonite modified with 3 different QACs, which include Tetramethylammonium (TMA), Hexadecyltrimethylammonium (HDTMA) and Benzyldimethylhexadecylammonium (BDHDMA). The Batch adsorption results demonstrated that HDTMA-Montmorillonite and BDHDMA-Montmorillonite are better adsorbent than the precursor and TMA-Montmorillonite. The adsorption affinity of Montmorillonite towards humic acid increased with an increase of the QACs Carbon atoms in the organo-clays. Solution of TMA, HDTMA and BDHDMA were used separately as the modifying agent in the preparation of organoclays. The synthesis was carried out in a batch reactor with 0.5 litres of each QAC solution. The adsorption isotherms were conducted using a batch equilibration technique in a 0.5 litre conical flask by varying the initial humic acid concentration while maintaining the amount of adsorbent clay. A control sample with no adsorbent was also prepared. In addition, comparative adsorption experiments were conducted with the untreated Montmorillonite as a reference material. The flasks were closed and shaken overnight to allow equilibrium to be achieved.

Tsau and Aarne (1998) studied the impact of addition of lime to a mixture of montmorillonite clay, and sand. The results of laboratory experiments conducted by them involving test of hydraulic conductivity against Zinc Chloride solution, suggest of changes in chemical & physical properties of clay because of lime addition. They found that the hydraulic conductivity of lime clay mixture increases with increase in percentage of lime to an extent and decreases thereafter with further lime addition. Lime has also been found to increase mixture’s puncture resistance. They concluded that the lime addition to clay results in increase in hydraulic conductivity with increase in lime content upto about 5-10%. With lime content greater than this hydraulic conductivity decreases. Higher amount of lime in clay results in greater amount of Zinc capture delaying Zinc breakthrough. Lime addition to clay results in increase in puncture resistance of landfill liner which increases linearly with increase in lime percentage. Possibility of destruction of clay due and consequent increase in its hydraulic conductivity to acidic leachate, can be avoided by lime addition to clay.

Chapter 3

Contaminant Transport Model for Landfill

CHAPTER 3

CONTAMINANT TRANSPORT MODEL FOR LANDFILL

3.1 Introduction

In order to protect the contamination of groundwater due to landfill leachates, the waste materials needs to be placed in a engineered landfill. The design of engineered landfills is regulated by relevant agencies to ensure that the pollution prevention criteria in a region is met with. To demonstrate the likelihood of compliance with regulations, environmental engineers make routine recourse to mathematical models of proposed landfill. One outstanding feature of the modeling approach is the capacity for predictions to be made well into the future thereby determining the likely impacts of current practice on the environment and future generations.

Evaluation of the design for a waste disposal facility involves making a quantitative prediction of the potential impact of the waste on groundwater quality, keeping in mind that under most circumstances involving contaminant movement through a soil liner and into the aquifer, the best one can do is to predict the trends and a likely range of concentrations at any given point in space and time. There are four aspects of any attempt to make quantitative predictions, viz. the need to identify the controlling mechanisms, formulate or select the theoretical model, determine the relevant parameters, solve the governing equation. When dealing with contaminant transport through saturated clayey landfill liners the primary transport mechanism are advection and diffusion/dispersion.

3.2 Mechanisms of Contaminant Migration

In the event of leachate mounding over the top of liner, the leachate from landfills tends to migrate into the barrier/ bottom aquifer carrying away with it the contaminants leached from landfill. There are various mechanisms which account for the migration of leachate contaminants from landfill into the barrier. Modeling of contaminant migration involves taking into account the various processes involved in it like advection, diffusion/ dispersion.

3.2.1 Advection

As leachate moves down into the barrier, it carriers contaminants along with it. Thus when dealing with contaminants in leachate, the mass of contaminant transported by advection per unit area per unit time is given by $f=nvc= v_a c$, where n is the effective porosity of soil, v is groundwater velocity (seepage velocity), v_a is Darcy velocity, c is concentration of contaminant at the pint and time of interest. Thus the total mass transported from a landfill into a barrier up to some specific time t, is obtained by summing the mass flux with respect to τ.

$$m_a = A_0 \int_0^t nvc\,d\tau \qquad \text{……………3.1}$$

Where m_a is mass of contaminant transported from the landfill by advection, A_0 is the cross sectional area of landfill through which contaminant is passing.

3.2.2 Diffusion

Diffusion involves the movement of leachate contaminants under concentration gradients i.e. difference of concentration of contaminants in leachate and in barrier/ aquifer. Diffusive transport in one dimension can be expressed as:

$$f = -nD_e \frac{\partial c}{\partial z} \qquad \text{...............3.2}$$

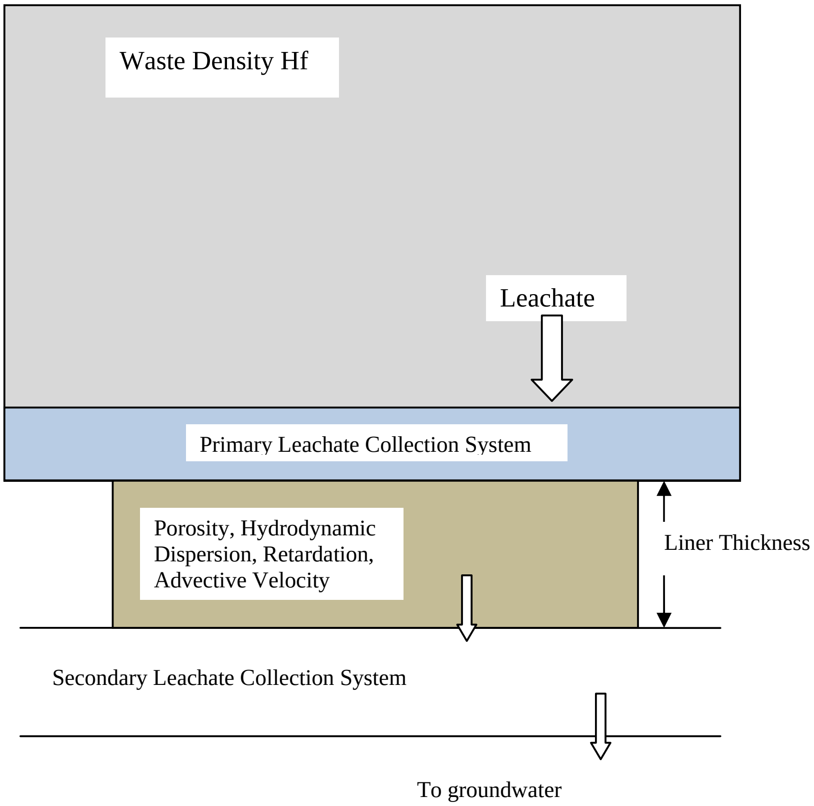

Fig. 3.1 Schematic Diagram of Contaminant Migration from Landfill

Where n is effective porosity of barrier material, D_e is the effective molecular diffusion coefficient, ∂c/∂z is the concentration gradient. The negative sign arises from the fact that the contaminants move from high to low concentrations. The total mass of contaminant, m_d, transported out of landfill by diffusion up to some specific time t, is obtained by integrating the above equation

$$m_d = A_0 \int_0^t \left(-nDe \frac{\partial c}{\partial z}\right) d\tau \qquad \text{................3.3}$$

3.2.3 Advective diffusive transport

As discussed before, in the absence of diffusion, contaminant would be transported at the advective velocity of leachate into the barrier. However, diffusion cannot be neglected. If the direction of diffusive transport is the same as the direction of advective flow, it will increase the amount of contaminant transport and decrease the time it takes for contaminant to move to a given point away from the source. For the case of advective diffusive transport the mass flux, f of the contaminants into the barrier can be expressed as

$$f = nv_x c - nD_h \frac{\partial c}{\partial z} \qquad \text{................3.4}$$

And the total mass, m transported from the landfill up to specific time t is given by

$$m = A_0 \int_0^t \left(nv_x c - nD_h \frac{\partial c}{\partial z}\right) d\tau \qquad \text{.................3.5}$$

When contaminant migration is associated with relatively high flows (as in many aquifers), there is a third transport mechanism to be considered viz. mechanical dispersion. Dispersion process involves mixing of the contaminants that occurs due to local variations in the flow velocity of the groundwater. The dispersion of leachate contaminants also involves mixing and spreading of the contaminants due to non-homogeneity of barrier/aquifer. Although, this mechanism is totally different from the diffusion process, for most practical applications, it can be mathematically modeled in the same way, hence the two processes are often lumped together as composite parameter, D, called the coefficient of hydrodynamic dispersion $D_h=D_e+D_{md}$, where D_e is the effective molecular diffusion coefficient for the contaminant species of interest, D_{md} is the coefficient of mechanical dispersion. When dealing with transport through intact clayey soil, diffusion will usually control the parameter D and dispersion is negligible. In aquifers, the opposite tends to be true and dispersion tends to dominate. It is often convenient to model the dispersive process as linear function of velocity. Thus $D_{md}=\alpha v$, where α is dispersivity. The dispersivity tends to be scale dependant.

3.2.4 Sorption

As leachate moves down into the barrier, a part of the contaminants carried in it are sorbed by the barrier media which is reflected as the retardation of contaminants by barrier media. The process of sorption of contaminants by the barrier can be modeled as being linear and reversible, and so the mass of contaminant removed from solution, S, is proportional to the concentration in solution, $S=K_dc$, where S is the mass of solute removed from solution per unit mass of solid, K_d is the partitioning or distribution coefficient, c is the equilibrium concentration of solute.

At high concentrations, sorption process is non linear and the two commonly used sorption isotherms are Langmuir and Freundlich isotherms. The Langmuir isotherm has a good theoretical basis, but, unfortunately, is not always adequate for describing sorption processes. The Freundlich isotherm provides an empirically based alternative model which sometimes provides a better quantitative description of sorption. It can be written as $S=K_fC^b$, where K_f and b are empirically determined constants. The parameters K_f and b are best determined by performing batch sorption tests. The sorption of contaminants carried by leachate is modeled as retardation factor, represented by R_f, and is written as

$$R_f = 1 + \frac{\rho K_f C^{b-1}}{n} \qquad \text{...............3.6}$$

3.3 Governing Differential Equation

The mass flux f of the contaminant species, which is transported per unit area of the landfill, per unit time due to advective diffusive transport can be expressed as

$$f = nv_x c - nD_h \frac{\partial c}{\partial z} \qquad \text{....................3.7}$$

Considering the conservation of mass within any small region inside the barrier, the change in concentration of contaminant in the small region within the barrier, with time is given by

$$n\frac{\partial C}{\partial t} = -\frac{\partial f}{\partial z} - \rho\frac{\partial S}{\partial t} - n\lambda c \qquad \text{....................3.8}$$

Which simply says that the increase in contaminant concentration within a small region inside the barrier below landfill is equal to the increase in mass due to advective-diffusive transport minus the decrease in mass due to sorption minus the decrease in mass due to first order decay processes.

Substituting for muss flux *f*

$$n\frac{\partial C}{\partial t}=\left(nD\frac{\partial^2 C}{\partial z^2}-nv\frac{\partial C}{\partial z}\right)-\rho K_d\frac{\partial C}{\partial t}-n\lambda c$$ ……………..3.9

The process for first order or sorption may be modeled as retardation factor R_f. The above equation in that case can be rearranged to have the form

$$\boxed{\frac{\partial C}{\partial t}=\frac{D_h}{R_f}\frac{\partial^2 C}{\partial z^2}-\frac{v}{R_f}\frac{\partial C}{\partial z}}$$ ………………..3.10

Where $R_f=1+[\rho K_f C^{(b-1)}]/n$, is the retardation coefficient.

Eq. 3.10 forms the mathematical model representing transport of contaminants from a landfill. The solution of Eq. 3.10 shall yield the variation of concentration of contaminant species of interest with respect to time and depth. The solution of Eq. 3.10 can only be obtained against suitable boundary conditions to ape the field conditions of landfill.

3.4 Boundary Conditions

To obtain a unique solution of Eq. 3.10 additional information about the physical state of the process is required. This information is supplied in the form of initial and boundary conditions. For steady-state problems, only boundary conditions are required, whereas for transient problems, boundary and initial conditions must be specified. Mathematically, the boundary conditions include the geometry of the boundary and the values of the dependent variable or its derivative normal to the boundary.

3.4.1 Finite mass of contaminant

In many practical situations, the mass of contaminant within a landfill is limited and mass will be reduced as the contaminant is transported into landfill barrier. The concentration of potential contaminants in a landfill generally increases during the operation of landfill facility, reaches a peak, and then declines. The increase in concentration is related to leaching of contaminants, chemical and biological processes which generate chemical species of interest due to synthesis, or breakdown of existing species. Similarly the decrease in concentration of leachate contaminants with time may be related to removal of contaminants in the form of leachate, from the landfill, chemical and biological processes which result in precipitation and/or synthesis or breakdown of the chemical species of interest into other chemical forms.

For the design of barrier systems it is generally not practical to model the details of leaching processes or of any associated chemical or biological processes. However, reasonable engineering approximations can be made which will allow the designer to obtain some insight into the potential impact of finite mass of contaminant. Thus for the purposes of performing design calculations, it is often conservative to assume, that the concentration of contaminant of interest reaches a peak, c_0, instantaneously, and all of the mass of contaminant species m_{tc} is in solution at the time when peak concentration occurs. The mass of the contaminant available for transport into the soil can be represented in terms of the peak concentration, c_0, and the reference height of leachate, H_r, or the equivalent height of leachate H_f. H_r, may be defined for each contaminant species of interest and refers to the volume of fluid (per unit area of landfill) that, at a concentration c_0, would contain the total mass m_0 of that contaminant species which could be released either for transport or collection. The equivalent height of leachate H_f, refers to the portion of mass that is available for transport into hydrogeologic system. It does not include the contaminant that is or is expected to in solid immobile form, or contaminant that is released in gas phase. The equivalent height of leachate has similar definition except that it corresponds to that portion of mass that is available for transport into the hydrogeologic system. Thus the essential difference between H_r and H_f is that the H_r includes the mass collected by the leachate collection system, whereas H_f excludes this mass. Considering the conservation of mass within the source solution, we can write

$$\begin{bmatrix}\text{Mass of contaminant}\\ \text{Within source at}\\ \text{Time t}\end{bmatrix} = \begin{bmatrix}\text{Initial mass of}\\ \text{contaminant}\\ \text{with source}\end{bmatrix} - \begin{bmatrix}\text{Mass of contaminant}\\ \text{transporated into}\\ \text{the soil}\end{bmatrix} - \begin{bmatrix}\text{Mass of contaminant}\\ \text{lost due to first order}\\ \text{decay processes}\end{bmatrix}$$

$$m_t = m_{TC} - m - m_{DC} \qquad \text{……………………3.11}$$

Now, mass of the contaminant transported into the soil from landfill can be written as

$$m = A_0 \int_0^t \left(n v_a c - n D_h \frac{\partial c}{\partial z} \right) d\tau \qquad \text{……………3.12}$$

Where A_0 is the area of landfill, v_a is seepage velocity, n is porosity, D_h, is hydrodynamic dispersion coefficient. Thus Substituting for mass of contaminant transported into the soil mass the above equation can be re-written as

$$m_t = m_{TC} - A_0 \int_0^t \left(n v_a c(\tau) - n D_h \frac{\partial c}{\partial z} \right) d\tau - m_{DC} \qquad \text{……………..3.13}$$

Where $m_t = A_0 H_r c(t)$ is the mass of contaminant in source at time *t*. $m_{TC} = A_0 H_r c_0$ is the initial mass of contaminant in landfill, m_{DC} is the mass lost due to first order decay process, *c(t)*n is the concentration of contaminant in landfill at time *t*, *A₀* is the area of landfill through which contaminant can pass into soil. Above equation models both the total mass of contaminant in landfill and removal of contaminant from the landfill. An alternative approach is to reduce the total mass of contaminant available for transport into the groundwater, by excluding the portion of mass which will be collected by a primary leachate collection system. For this case the finite mass boundary condition can be written in terms of equivalent leachate height. This gives rise to equation

$$c_T(t) = c_0 - \frac{1}{H_f}\int_0^t f_T(c,\tau)\,d\tau \qquad \ldots\ldots\ldots\ldots\ldots\ldots 3.14$$

The second term in Eq. 3.14 can be written as

$$f\left(c,\tau\right) = nv_x c_T\left(0,t\right) - nD_h \frac{\partial c_T\left(0,t\right)}{\partial x} = n\left[v_x c_T\left(0,t\right) - D_h \Phi\left(0,t\right)\right] \qquad \ldots.3.15$$

$$\int_0^\tau f\left(c,\tau\right) d\tau = n\left(v_x \sum_{t=0}^{t} c_T(0,t)\Delta t - D_h \sum_{t=0}^{t} \Phi\left(0,t\right)\Delta t\right) \qquad \ldots\ldots 3.16$$

Substituting 3.16 in 3.14,

$$c_T(0,t) = c_0 - \frac{n}{H_f}\left(v_x \sum_{t=0}^{t} c_T(0,t)\Delta t - D_h \sum_{t=0}^{t} \Phi\left(0,t\right)\Delta t \right) \quad .....3.17$$

The mass transport of contaminants from the bottom of landfill is determined by solving the governing equation Eq. 3.10 subject to upper boundary condition represented by Eq. 3.17, and initial and lower boundary conditions as under:

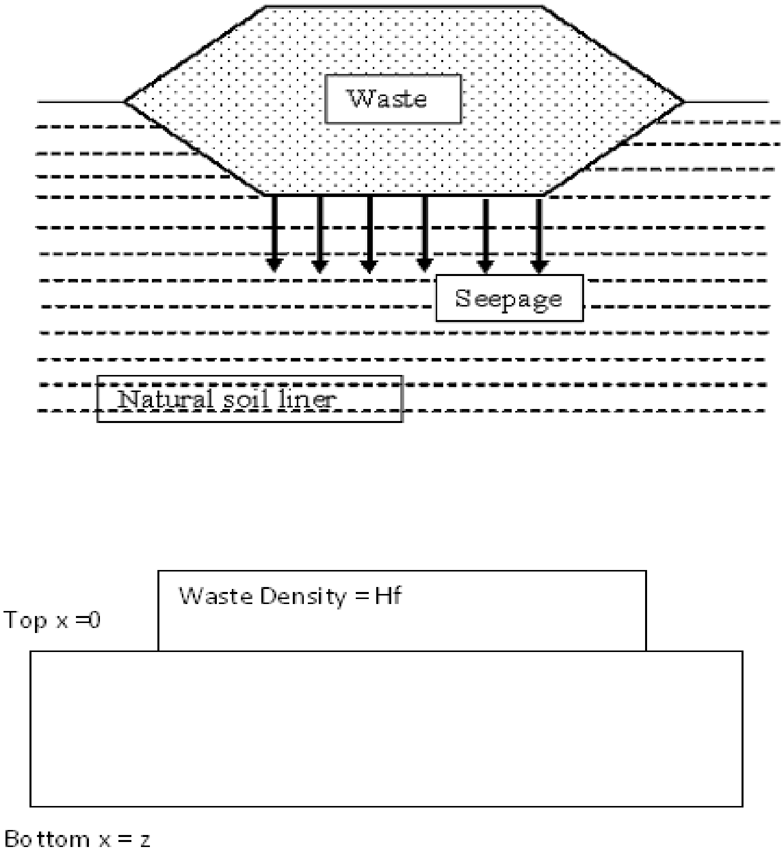

Fig. 3.2 Schematic Diagram of Landfill and Liner

Initial Condition $C_t(x,0)=0$ 3.18

Lower Boundary Condition $C_t(\infty, t)=0$ 3.19

The contaminant transport model represented by Eq. 3.10 was solved with a numerical scheme using Finite Difference Method, subjected to initial and lower boundary conditions represented by Eq. 3.18, and 3.19 and upper boundary condition represented 3.17. The details of solution are discussed below.

3.5 Model Solution

The solution to the contaminant transport model for landfill represented by Eq. 3.10, subject to boundary conditions represented by Eq. 3.17, 3.18, and 3.19 was implemented using the numerical method, Explicit Finite Difference Method (FDM) with upwind correction. Eq. 3.10 describing the contaminant transport model for landfill include terms representing derivatives of continuous variables. Finite difference methods are based on the approximation of these derivatives (or slopes of curves) by discrete linear changes over small discrete intervals of space or time. If the intervals are sufficiently small, then all of the linear increments will represent a good approximation of the true curvilinear surface. For every node of the grid we will have a separate difference equation, each of which contains only one unknown variable. Thus, these equations can be solved explicitly, and the method thus is called as Explicit Finite Difference Method. Explicit finite-difference equations are thus simple and straightforward to solve, but have stability criteria associated with them. That is, if time increments are too large, small numerical errors or perturbations may propagate into larger errors at later stages of the computations. Thus for the solution of Eq. 3.10, domain of the problem of variation of concentration of contaminant with respect to time and depth below landfill and inside the barrier was descritized into suitable number of nodes of space and time. The descretization of domain is shown in Fig. 3.3. Governing Eq. 3.10 in the finite general difference with upwind correction form is written as

$$\frac{C_i^{m+1}-C_i^m}{\Delta t}+\frac{1}{R_f}\alpha v_x\frac{C_{i+1}^{m+1}-C_{i-1}^{m+1}}{2\Delta x}+\frac{1}{R_f}(1-\alpha)v_x\frac{C_{i+1}^m-C_{i-1}^m}{2\Delta x}-$$
$$\frac{1}{R_f}\alpha D_x\frac{C_{i+1}^{m+1}-2C_i^{m+1}+C_{i-1}^{m+1}}{(\Delta x)^2}-\frac{1}{R_f}(1-\alpha)D_x\frac{C_{i+1}^m-2C_i^m+C_{i-1}^m}{(\Delta x)^2}=0 \quad ..3.18$$

Where 0≤α≤1. The case of setting α=0, results in explicit finite difference scheme.

$$\frac{C_i^{m+1}-C_i^m}{\Delta t}+\frac{1}{R_f}v_x\frac{C_{i+1}^m-C_{i-1}^m}{2\Delta x}-\frac{1}{R_f}D_x\frac{C_{i+1}^m-2C_i^m+C_{i-1}^m}{(\Delta x)^2}=0 \quad .....3.19$$

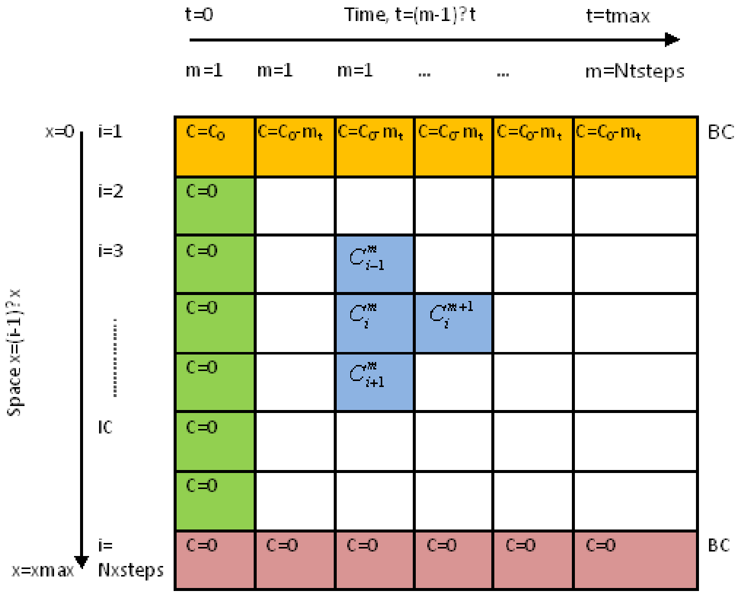

Fig. 3.3 Schematic Diagram of Finite Difference Method

$$\frac{C_i^{m+1}-C_i^m}{\Delta t}+\frac{1}{R_f}v_x\frac{C_i^m-C_{i-1}^m}{\Delta x}-\frac{1}{R_f}D_x\frac{C_{i+1}^m-2C_i^m+C_{i-1}^m}{(\Delta x)^2}=0 \qquad \text{……3.20}$$

Multiplying all the terms by Δ*t*, we get

$$C_i^{m+1}-C_i^m+\frac{v_x\Delta t}{R_f\Delta x}\left(C_i^m-C_{i-1}^m\right)-\frac{D_x\Delta t}{R_f(\Delta x)^2}\left(C_{i+1}^m-2C_i^m+C_{i-1}^m\right)=0 \qquad \text{…….3.21}$$

Solving for C_i^{m+1}

$$C_i^{m+1}=C_i^m-\frac{v_x\Delta t}{R_f\Delta x}\left(C_i^m-C_{i-1}^m\right)+\frac{D_x\Delta t}{R_f(\Delta x)^2}\left(C_{i+1}^m-2C_i^m+C_{i-1}^m\right)=0 \qquad \text{……..3.22}$$

Writing $\kappa_1=\frac{1}{R_f}\frac{v_x\Delta t}{2\Delta x}$ and $\kappa_2=\frac{1}{R_f}\frac{D_x\Delta t}{(\Delta x)^2}$, one gets

$$C_i^{m+1}=C_i^m-(1-2\kappa_1-2\kappa_2)C_i^m+\kappa_2C_{i+1}^m+(2\kappa_1+\kappa_2)C_{i-1}^m=0 \qquad \text{…….3.23}$$

The method was implemented by marching the solution forward at the interior nodes, bringing with it the effects of the initial condition and the boundary nodes. For the implementation of the above solution in MatLab, the domain of problem is descritized in suitable no. of nodes for time and space, in such a way that $\left(\frac{1}{R_f}\frac{D_x\Delta t}{(\Delta x)^2}+\frac{1}{R_f}\frac{v_x\Delta t}{2\Delta x}\right)\leq\frac{1}{2}$.

Thus the entire domain was divided in say, N_{tsteps} =T/Δ*t*, and N_{xsteps}=z/Δ*x*. Initial and boundary conditions are implemented by keeping C_i^m at the beginning of solution to be zero everywhere along the entire depth of domain. Thus for the implementation of initial condition, keeping C_i^m to be zero everywhere, one has

$$C_i^m = C_2^m = C_3^m = C_4^m = C_5^m = C_6^m = C_7^m = C_8^m \ldots\ldots\ldots\ldots\ldots\ldots C_{Nxsteps}^m$$

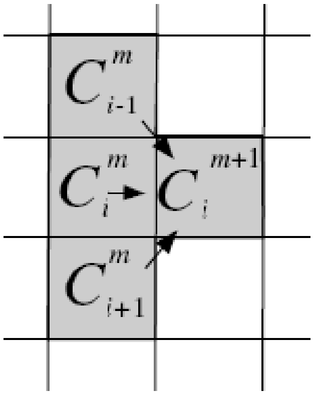

Fig. 3.4 FDM Node

At the next time step, the solution is marched from *x=2* to N_{xsteps}-1, such that

$$C_2^2 = C_2^1 - (1 - 2\kappa_1 - 2\kappa_2)C_2^1 + \kappa_2 C_3^1 + (2\kappa_1 + \kappa_2)C_1^1 = 0$$

$$C_3^2 = C_3^1 - (1 - 2\kappa_1 - 2\kappa_2)C_3^1 + \kappa_2 C_4^1 + (2\kappa_1 + \kappa_2)C_2^1 = 0$$

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$$C_{Nxsteps-1}^2 = C_{Nxsteps-1}^1 - (1 - 2\kappa_1 - 2\kappa_2)C_{Nxsteps-1}^1 + \kappa_2 C_{Nxsteps}^1 + (2\kappa_1 + \kappa_2)C_{Nxsteps-2}^1 = 0$$

Similarly, for the next cycle of time step, one would write the equations as,

$$C_2^3 = C_2^2 - (1-2\kappa_1 - 2\kappa_2)C_2^2 + \kappa_2 C_3^2 + (2\kappa_1 + \kappa_2)C_1^2 = 0$$

$$C_3^3 = C_3^2 - (1-2\kappa_1 - 2\kappa_2)C_3^2 + \kappa_2 C_4^2 + (2\kappa_1 + \kappa_2)C_2^2 = 0$$

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$$C_{Nxsteps-1}^3 = C_{Nxsteps-1}^2 - (1-2\kappa_1 - 2\kappa_2)C_{Nxsteps-1}^2 + \kappa_2 C_{Nxsteps}^2 + (2\kappa_1 + \kappa_2)C_{Nxsteps-2}^2 = 0$$

Similarly, for the last cycle of time step, one would write the equations as,

$$C_2^{Ntsteps} = C_2^{Ntsteps-1} - (1-2\kappa_1 - 2\kappa_2)C_2^{Ntsteps-1} + \kappa_2 C_3^{Ntsteps-1} + (2\kappa_1 + \kappa_2)C_1^{Ntsteps-1} = 0$$

$$C_3^{Ntsteps} = C_3^{Ntsteps-1} - (1-2\kappa_1 - 2\kappa_2)C_3^{Ntsteps-1} + \kappa_2 C_4^{Ntsteps-1} + (2\kappa_1 + \kappa_2)C_2^{Ntsteps} = 0$$

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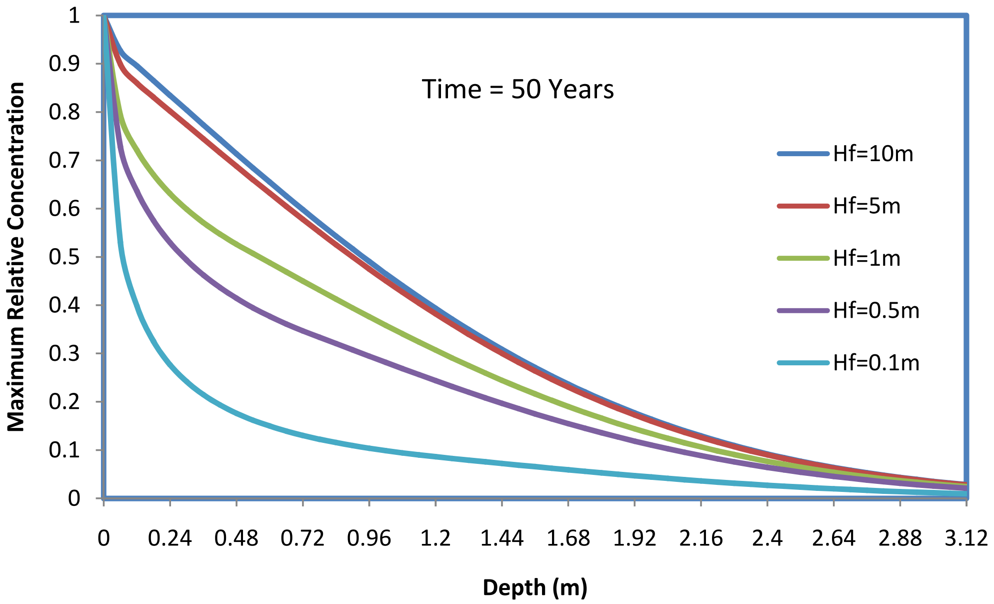
$$C_{Nxsteps-1}^{Ntsteps} = C_{Nxsteps-1}^{Ntsteps-1} - (1-2\kappa_1 - 2\kappa_2)C_{Nxsteps-1}^{Ntsteps-1} + \kappa_2 C_{Nxsteps}^{Ntsteps-1} + (2\kappa_1 + \kappa_2)C_{Nxsteps-2}^{Ntsteps-1} = 0$$

The solution of the model results in a matrix which provides spatial and temporal variation of concentration of the contaminant below landfill. One such result of the simulation of the model is shown in Fig. 3.5, which shows the variation of maximum relative concentration of conservative contaminant below landfill after a period of 50 years. Depending upon the need the simulation of the model may be run for the desired time period and the results may be obtained either with respect to depth after a certain period of time or at a particular depth with respect to time.

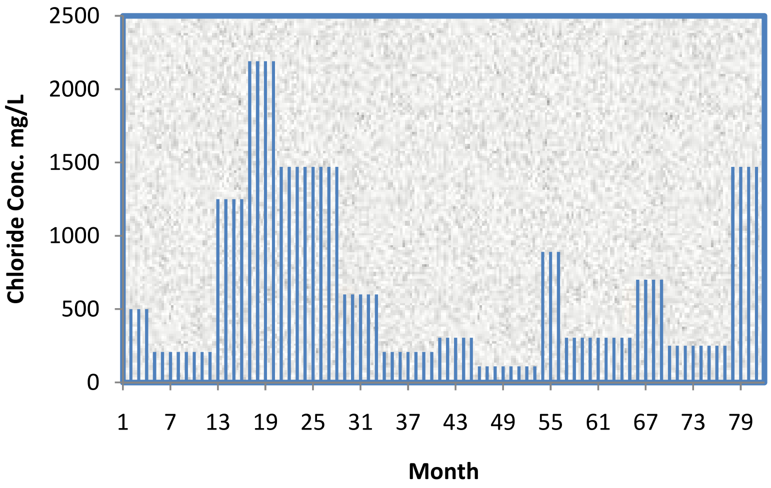
Fig. 3.5 Results of the Simultaion Run of Model



3.6 Validation of Model

Model developed herein was tested for two cases of field data. In the first case, field data of Munro et al. (Munro et al. 1997) for a landfill at New Brunswick was used to validate the numerical model. The field profile of chloride in the media was compared with the numerical model developed in the study. The mechanisms controlling the contaminant transport in the shallow till deposit at the landfill site were predominantly advection and advection-dispersion for the conservative contaminant viz. chloride. However, since the shallow zone of the till comprised hydraulically active fractures, there was a significant advective transport. At greater depths the dominant process controlling the downward migration of landfill leachate was molecular diffusion. In their investigation, the hydraulic gradients in the till underlying the landfill waste was determined by them using data from monitoring wells, and from comparison of the values obtained for horizontal and vertical hydraulic gradients. It was concluded by them, that the movement of contaminant was predominantly vertical. To calibrate their field data, Munro et al. (Munro et al. 1997) adopted a process based approach for estimating the temporal concentrations of chlorides at the waste till interface. This concentration in turn was adopted as the source concentration (input) in their mathematical model. They developed two types of solute transport model. The upper portion of till was found to have fracture spacing of 5mm and thus could also be modeled using porous media approach. Likewise, McKay et al. (McKay et al. 1993) also suggested that for weathered upper zone in fractured clay till with fracture spacing of 130mm, the fractured media could be modeled as an equivalent porous media. Thus the governing equation of solute transport through porous media was found to be appropriate for mathematical modeling in the upper till. Moreover, the maximum depth of solute migration was found to be beyond the depths sampled at the bore wells and there was not sufficient information available for greater depths in till.

Fig. 3.6 Chloride Concentration Histogram



Also due to the possible uncertainties in the chloride source function, comparison of the field data and numerical results were considered to be good if the model simulation adequately fit the steeply declining concentrations in the top one meter of the profile (Munro et al. 1997). For greater depths (around 1.5m below the landfill), their model results did not agree with the observed data and the sharp localized concentration changes identified beyond 1.5 were attributed to the variations in the source concentration within the landfill and/or due to unidentified changes in local geochemistry. The domain for the model developed in this study is thus restricted to this depth. The source concentration for chloride is shown as histogram of estimated chloride concentration (Fig. 3.6) in leachate since the inception of the landfill in 1985 (Munro et al. 1997). The field parameters obtained from field monitoring are listed in Table 3.1 and the simulated and observed chloride concentration at different depths are shown in Table 3.3. Similarly for the second case, field data of organic dissolved carbon at the same location was compared with the simulated model data. The contaminant was non conservative and the model took into account advection, dispersion-diffusion, and retardation of the contaminant as it moved vertically down below the landfill. In this case too, advection and mechanical dispersion are more important than molecular diffusion for shallow depths of the till.

Table 3.1 Model Parameters for Chloride (Munro et al. 1997)

S.No.	Model Parameter	Unit	Value
1	Depth	m	4.0
2	Effective Molecular Diffusion Coefficient	m^2/yr	0.02
3	Porosity		0.28
4	Retardation factor (R)		1.0
5	Advective velocity	m/yr	0.7

Table 3.2 Model Parameters for DOC (Munro et al. 1997)

S.No.	Model Parameter	Unit	Value
1	Depth	m	4.0
2	Effective Molecular Diffusion Coefficient	m^2/yr	0.02
3	Porosity		0.28
4	Retardation factor (R)		1.5
5	Advective velocity	m/yr	0.7

Table 3.3 Simulated and Observed Chloride Concentration (Time 6.75 years)

Depth	Observed Chloride Conc.* (mg/L)	Simulated Chloride Conc. (mg/L)
0.18	1250	1055
0.38	814	744
0.54	680	506
0.6	500	453
1.2	500	403
1.38	435	397
1.5	355	390
2.1	460	490
2.7	600	707
3.06	636	773

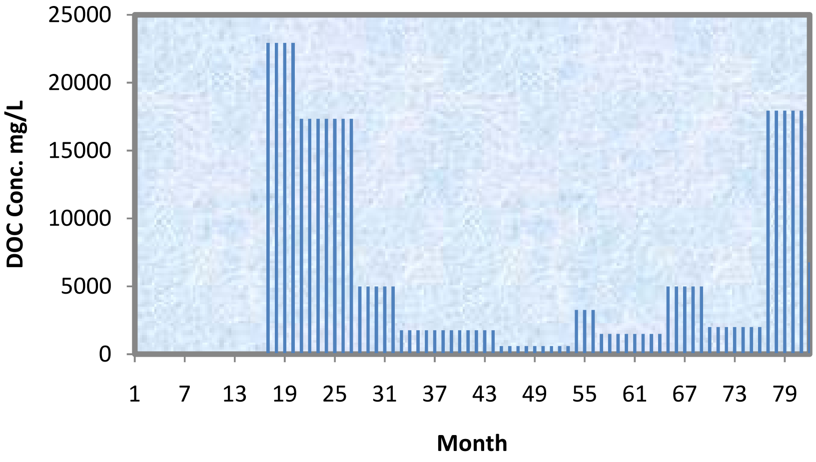
*Munro et al. 1997

Table 3.4 Simulated and Observed DOC Concentration (Time 6.75 years)

Depth	Observed DOC Conc.*	Simulated DOC Conc. (mg/L)
0.16	10000	12670
0.3	8040	10444
0.4	5000	7075
0.54	4200	4585
0.65	3700	3436
0.76	2850	2886
0.82	2760	2790
1.08	2860	2752

*Munro et al. 1997

Fig. 3.7 Histogram : DOC Concentration



The source concentration for Dissolved Organic Carbon (DOC) is shown as histogram of estimated DOC concentration (Fig. 3.7) in leachate since the inception of the landfill in 1985 (Munro et al. 1997). The simulation was done for the retardation factor of 1.5. The results of simulation obtained for this case are compared with the observed field data. Simulated results match well with the observed values by Munro et al. (Munro et al. 1997). Field parameters for dissolved organic carbon are shown in Table 3.2 and the simulated and observed DOC concentration at various depths is shown in Table 3.4.

3.7 Sensitivity Analysis of the Model

A sensitivity analysis of the model was carried out to determine the sensitivity of the model results. For the purpose of sensitivity analysis, simulation of the model was carried out for different values of the parameters viz. advective velocity, and dispersion coefficient. Analysis was carried out for the time period of 50 years and equivalent height was taken as 10m. Advective velocity was taken as 0.1, 0.2, 0.3, 0.4, and 0.5 m/year. Hydrodynamic dispersion coefficient was taken as 0.01, 0.05, and 0.15 m^2/year.

The variation of maximum relative concentration of the conservative contaminant with respect to permeability after 50 years of period is shown in Fig.3.8. It can be seen from the figure that sharp increases in maximum relative concentration of the contaminant is expected to occur for permeability value of 0.0075 m/yr and above. Fig. 3.9 shows the variation of maximum relative concentration with respect to depth after 50 years of period and with respect hydrodynamic dispersion coefficient. As can be seen from this figure variation in the concentration of the contaminant species with respect to depth tends to stabilize with increase in hydrodynamic dispersion coefficient. A sharp increase of the stabilization of contaminant concentration can be seen for the hydrodynamic dispersion coefficient of 0.15 m^2/yr.

Fig. 3.8 Variation of Maximum Relative Concentration with Respect to Permeability

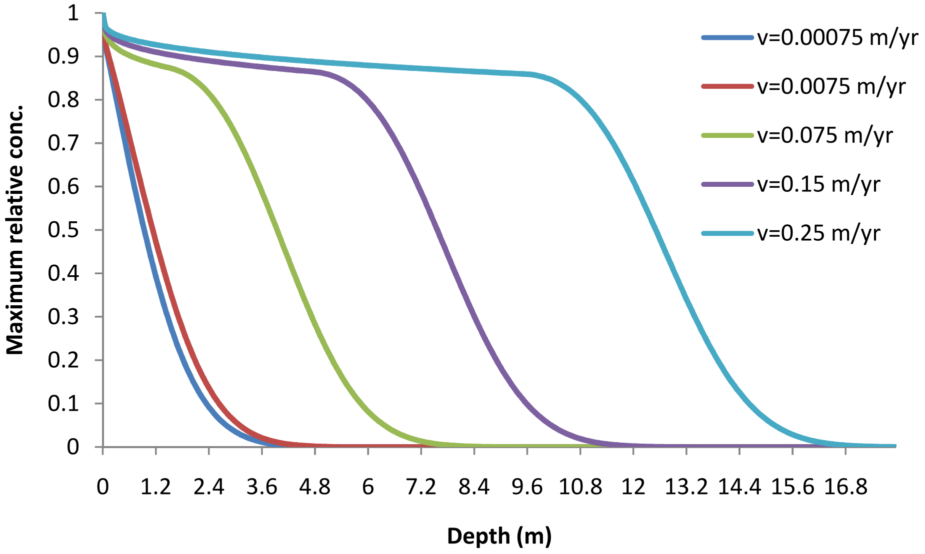
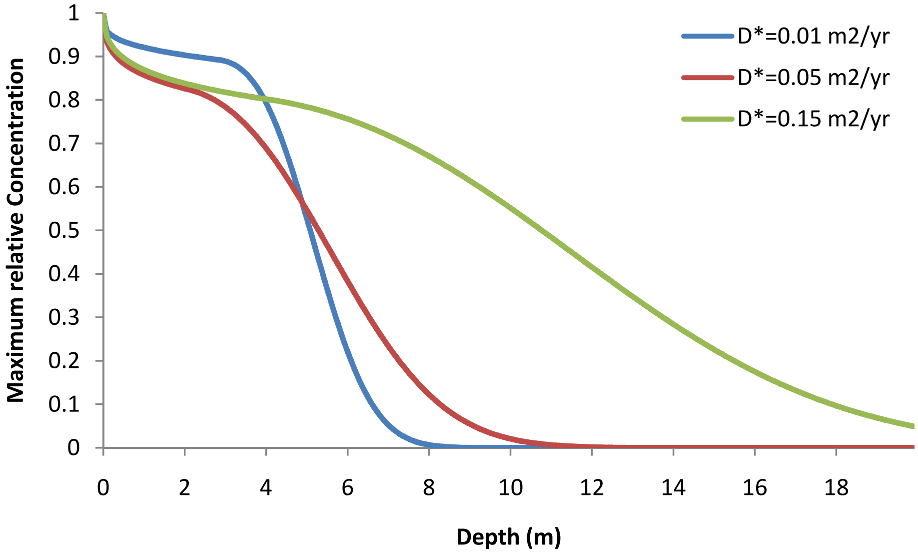


Fig. 3.9 Variation of Maximum Relative Concentration with Respect to Hydrodynamic Dispersion Coefficient



3.8 Advantages of Present Model

The model developed in this study has several advantages over the existing softwares available. These advantages are as under:

1. The use of public domain softwares is not advisable for the determination of transport from landfills, as the problems are expected due to variations in mass transport that may occur because of variations in landfill operations; also the migration of contaminants through landfill barrier layers is quite different from contaminant migration in aquifers.

2. For the determination of contaminant transport from landfills, special boundary conditions, i.e., finite mass boundary conditions, a thin layer of secondary leachate system and variation of concentration due to infiltration of water needs to be incorporated, and such facilities do not exists in software available in public domain. The data preparation of general purpose solute transport model is tedious process for the case of landfill.

3. Semi analytic and analytic method like POLLUTE and MIGRATE, SUPER1D (Sudicky 1988) have limited application for the landfills because they offer semi analytic solutions to simple problems with steady state concentrations boundary conditions and simplified initial conditions. These softwares are however user friendly and could be efficient tool for simulation studies.

4. The analytical solution available in public domain can be used only when the boundary concentration is a constant concentration condition. Also, it is assumed in the analytic solution that the natural

clay barrier below the landfill is of infinite depth. The analytic solution could give erroneous result if used for cases of finite thickness of the barrier. Also, in numerical solution one can actually ape the field conditions, especially with respect to the time variant concentration condition at the top boundary as well as full flush boundary condition, or a time variant concentration condition at the bottom boundary.

The model developed herein, addresses shortcoming discussed as above exclusively for the simulation of contaminant transport from landfills.

3.9 Limitations of Present Model

Limitations of model developed in this study are due to following assumptions made in the formulation of model

1. One dimensional vertical advective diffusive transport in clayey barriers/ aquitards.
2. Advection and diffusion/dispersion are considered as the primary transport mechanisms considered for the development of model. The density effects have been assumed to be insignificant, thus the migration of concentrated light or dense non aqueous phase contaminants is not considered.
3. Sorption processes are linear and can be represented in terms of partition or distribution coefficient.
4. There is no chemical reaction between migrating chemical species.
5. The medium is assumed to be saturated since the water content used in compacted liners is on the wet side of the optimum water content value, very close to saturation water content.

Chapter 4

Application of Model for Bhalaswa Landfill Site at Delhi

CHAPTER 4

APPLICATION OF MODEL FOR BHALASWA LANDFILL SITE IN DELHI

4.1 Introduction

The model developed in this study was applied to determine the likely impact of migration of landfill leachate on the groundwater quality in its vicinity of actual landfill site for key water quality parameters. The landfill site selected for the application of the model was Bhalaswa Landfill located in Delhi. The methodology for the application of model consisted of collection and analysis of samples of landfill leachate and groundwater for key water quality parameters in the areas surrounding of Bhalaswa landfill. Suitable values of model parameters were adopted taking into account the subsurface geography of the region.

4.2 Solid Waste and its Disposal in Delhi

Delhi generates about 6,000 metric tonnes of solid waste every day which is likely to increase to 18,000 metric tonnes per year by 2021. The present per capita waste generation rate is about 500 gm per person per day, which is almost 5 times of the national average of per capita solid waste generation. The main waste generated in Delhi is from the markets for agricultural products, retail and commercial establishments, hospital and nursing homes, slaughterhouses, industries and construction and demolition activities. Solid waste management in Delhi is mainly based on the disposal of the waste through the three operational landfills located at Bhalaswa, Okhla, and

Gazipur. These sites do not have any infrastructure to collect landfill gas emissions. For conversion of biodegradable components of solid waste, MCD has set up anaerobic, semi-mechanical compost plant near Okhla in 1980, and manual aerobic method of composting is practiced at Bhalaswa. All of these sites fall under the category of uncontrolled solid waste disposal facility. Many of the tip faces are steep and unstable and methane generation in the uncovered waste piles is spontaneously igniting causing smoldering surface fires.

Fig. 4.1 Waste Disposal Sites in Delhi

4.3 Bhalaswa Landfill

Bhalaswa landfill is located in one of the most urbanized area in a North West corner of Delhi. The landfill site is located between latitudes 28°42′30′′N and 28°45′N and longitudes 77°07′30′′E and 77°11′54′′E. It is surrounded by localities such as Bhalaswa and Jehangirpuri. The Bhalaswa landfill site occupies 21.06 hectares of land out of which about 5 hectares of land has been allocated to compost plant. The landfill site was once used for sugar cane plantation. The landfill at present receives about 2500 tonnes per day of waste out of which about 700 tonnes per day is sent to compost plant. The landfill has already reached about 22m of height. The landfill is publicly owned and is managed by MCD. The agency is concerned with the operation and management of the site which consists mainly of activities relating to receipt of the waste, processing and burial, on-going site preparation and restoration. The weighing bridge system is run by a private agency under contract with MCD. The incoming wastes originate mainly from households and commercial areas, but there are also some wastes which are brought to the landfill site from industries. However, before being carried to the landfill, these wastes undergo compaction at transfer stations. The landfill site is approximately square in shape with an area of about 16.06 hectares.

The sanitary landfill is divided into a number of cells. On a particular day, any one cell is selected and all the dumping is done in that cell. A cell is demarcated by access road on its sides. Soil cover is given after about 3-4 feet of material has been dumped. The soil cover is usually of malba. One compartment of the landfill has been constructed so as to accept hazardous wastes likely to cause health and environmental problems. However, the hazardous wastes cell has not come into operation yet because the regulations concerning the types of wastes that should be accepted at the landfill have not been finalized yet.

As the landfill site has not been planned and constructed on the principles of sanitary landfilling practice, the leachate percolates through the garbage and the lower soil layers and eventually mixes with the ground water. People living in various localities around the Bhalaswa landfill site are concerned about the groundwater quality of the region. However, no major disease has been reported so far.

Leachate percolating from the landfill gets collected in a leachate pond from where it is fed off to adjoining drain. No recirculation of the leachate is done in the landfill site. This creates possible contamination issue for the groundwater. Gas collection system does not exist. In summer, the garbage catches fire. Even in present condition, smoke can be seen coming out of the dumps at various places. The working personnel complain of weak eyesight due to exposure to methane. Composting and manufacture of manure is done at a site adjacent to Bhalaswa landfill. Fresh garbage is at first kept at yard station where actual process of composting is carried out. Each MCD truck brings about 4 tonnes of garbage.

4.4 Hydrogeology of Study Area

The geology of the area around Bhalaswa landfill is mainly alluvium [Fig. 4.2]. Groundwater flow direction in the vicinity of the Bhalaswa landfill is shown in Fig. 4.3, and a vertical cross-section of unconfined aquifer is shown in Fig. 4.4. There are several small patches of aquifers shown in the figures, but scientifically all these comprise a single aquifer system in Delhi (CGWB 1995).

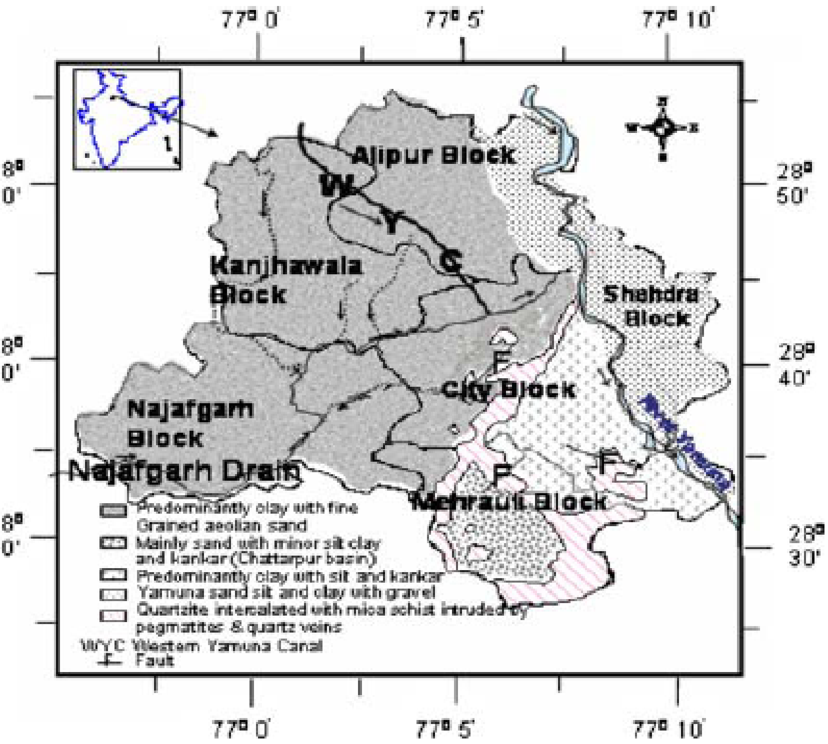


Fig. 4.2 Geology of Delhi (CGWB 2001)

The Delhi region is a part of the Indo-Gangetic Alluvial Plains, at an elevation ranging from 198–220 m above mean sea level. Lithologically, the area is transected by a quartzite rocky ridge, a prolongation of the Aravalli Hills extending along the southern border of Delhi and ending to the north on the west bank of the Yamuna River. The climate of Delhi tends to be of semiarid nature due to marked diurnal differences in temperature, high saturation deficit and low to moderate rainfall. The climate of Delhi is markedly periodic and is characterized by a dry and gradually increasingly hot season between March and June, a dry and cold winter from October to February and a warm, monsoon period from July to September. The average rainfall is 721 mm/year [Indian Meteorological Department (IMD) 1990–2004]. The minimum (average) and maximum (average) temperatures are 9.2 and 31.5 ^{o}C, respectively, with daily maximum temperatures during the hottest months commonly exceeding 41.2 ^{o}C.

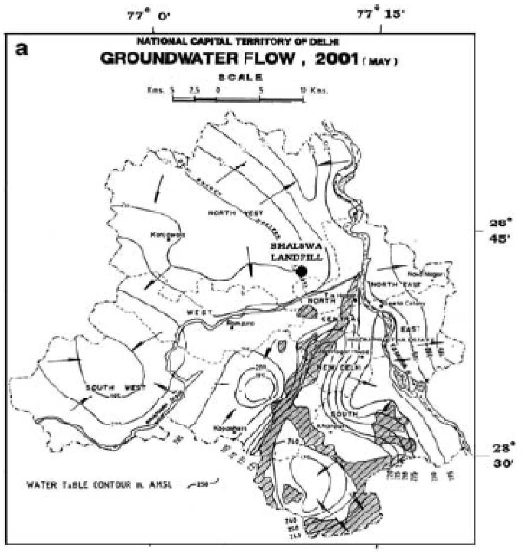


Fig. 4.3 Groundwater Flow Direction in Vicinity of Bhalaswa landfill (CGWB 2001)

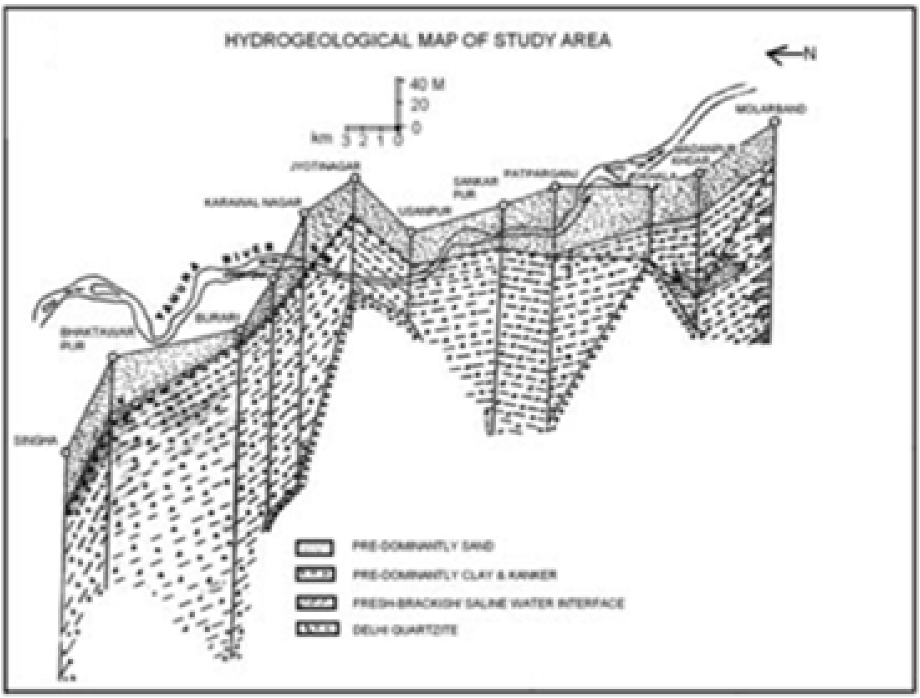


Fig. 4.4 Subsurface Geological section around Bhalaswa Landfill (CGWB 1995)

Physiographically, the region shows major variations, including the Delhi Ridge, which is an extension of the Aravalli hills consisting of quartzite rocks and extending from the southern part of the territory to the western bank of Yamuna for about 35 km. The alluvial formations overlying the quartzitic bedrock ate of different natures on either side of the ridge. The nearly closed Chattarpur alluvial basin covering an area about 48 km^2 is composed of the alluvium derived from the adjacent quartzite ridge alluvial plains on the eastern and western sides of the ridge and Yamuna flood plain deposits. These are of recent origin, and are also termed as newer alluvium. The newer alluvium is characterized by the absence of permanent vegetation because of periodic flooding and lack of kankar. The thickness of the alluvium overlying the bedrock increases in the direction away from ridge and reaches 300 m in the western parts of Nazafgarh, Kanjhawla block, and in the northern part of Alipur block. The thickness of the alluvium in the region of the Yamuna River on the east side of the ridge is about 165 m (CGWB 2003). Land utilization in Delhi has changed significantly over the years due to conversion of agricultural land for urban needs. A large part of the area has alkaline and saline soils with abundant calcrete depositions in places. The alkaline soils contain sodium bicarbonate and carbonate among the soluble salts, while the saline soils are impregnated with sodium chloride and sulphate as the main soluble salts. The distribution of these soils divides the study region into geographical subregions. The northwest region is covered by calcareous, silt clay loam; the northeast soils are calcareous, silt, and clay; and the southern parts are sandy-loam type.

4.5 Methodology

For the purpose of application and validation of contaminant transport model developed in this study, the likely impact of leachate migration from Bhalaswa landfill was determined by carrying out the simulation of model and the results of simulation run were compared with that of the results of analysis of groundwater samples from an area in the vicinity of landfill site.

Samples of leachate and groundwater in the area surrounding of landfill and at varying distances from the landfill location were collected and analysed for key water quality parameters. The results of such analysis are shown in Table 4.1.

Groundwater samples were collected from different locations at varying radial distances from the centre of landfill in the direction of groundwater flow. The subsurface geological section around Bhalaswa Landfill is shown in Fig. 4.4 Groundwater flow direction in the vicinity of Bhalaswa landfill is shown in Fig. 4.3. On the basis of subsurface geology of the region suitable model parameters were selected as shown in Table 4.2. Although data on amount of waste received as on today are available, the same are not available for the period beginning from the commencement of landfill facility. For the assessment of total amount of waste deposited, and thus the equivalent leachate height, the waste deposition has been assumed to have started with 700 tonnes per day in 1992, increasing by about 8% per annum, and reaching to about 2500 tonnes per day, as of now. Equivalent height of leachate has been calculated by assuming 0.2% chloride concentration in the total mass of waste deposited in landfill.

4.6 Results & Discussion

The results of analysis of leachate sample and groundwater samples at varying radial distances are shown in Table 4.1. The results of groundwater sample analysis clearly indicate the trend of reducing contaminant concentrations at increasing radial distances away from the landfill site for all the contaminants studied. The groundwater samples were collected from 3 locations in each radial distance group. The samples were collected from bore wells. The depth of all these bore wells in the area was around 10m from the ground surface.

Table 4.1 Analysis of Landfill Leachate and Groundwater Samples

Parameter	Concentration in landfill leachate	Concentration in groundwater sample at radial distance from landfill facility (mg/L)					
		≤100m	100-500m	0.5-1 km	1-1.5 km	1.5-2 km	2-2.5 km
Iron	8.2	2.1	1.4	1.11	1.1	1.03	0.7
Copper	0.8	0.1	0.07	0.06	0.05	0.02	0.01
Nickel	0.6	0.3	0.2	0.15	0.12	0.09	0.06
Zinc	3.2	0.8	0.5	0.3	0.03	0.02	0.02
Chloride	6840	1350.5	1140.24	986.2	640.2	448.4	236.3

Table 4.2 Model parameters for chloride Transport from Bhalaswa Landfill

S.No.	**Model Parameter**	**Unit**	**Value**
1	Time	year	50
2	Depth to the groundwater	m	10
3	Effective molecular diffusion coefficient	m^2/yr	0.02
5	Dispersivity	m	0.015
6	Porosity		0.40
7	Retardation factor (R)		1.0
8	Advective velocity	m/yr	0.5
9	Equivalent height of leachate	m	10
10	Area of landfill	ha	16.188
11	Chloride content in landfill mass	%	0.2
12	Maximum concentration of chlorides in landfill leachate	mg/L	6840

Fig. 4.5 Impact of Chloride Migration from Bhalaswa Landfill
Depth = 10m, Time 50 yrs

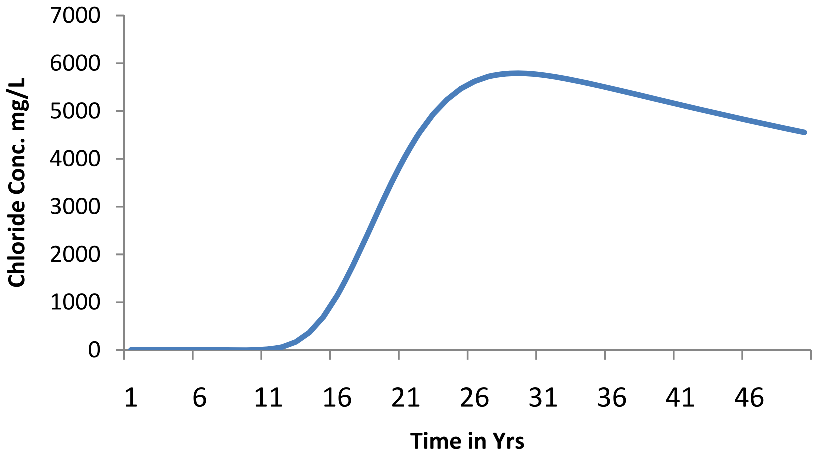


Table 4.3 Characterization Leachate from Bhalaswa Landfill

Parameter	**Concentration (mg/L)**
Total Suspended Solids	620
Total Dissolved Solids	20310
BOD5	4390
COD	10170
Total Organic Carbon	4980
Chlorides	6840
Fluoride	0.60
Total iron	8.20
Total Chromium	0.72
Lead	0.83
Zinc	3.2
Copper	0.80
Cadmium	0.92
pH	8.3

Simulated chloride concentration in the groundwater for 50 years of simulation period at a depth of 10m below the landfill facility, increases, reaches a peak, and then declines (Fig. 4.5). Simulated maximum chloride concentration reaches to a value of about 1678 mg/L in 16 years of operation of landfill facility for the selected model parameters. The observed concentration of chloride in groundwater sample within 75m radial distance of Bhalaswa landfill corresponding to a depth of about 10m has been found to be 1350.5 mg/L. The observed concentration of 1350.5 mg/L appears to be quite in agreement with the simulated concentration 1678.2 mg/L. However as the wastes is continued to be dumped because of unavailability of a new landfill site, the total mass of chloride available for leaching is expected to increase, which may further increase the concentration of chloride in groundwater. From results plotted it can be seen that the a concentration of about 1000 mg/L is likely to be present for a long period in the aquifer even, endangering the lives of people dependant on the water supply from surrounding area. Also with the passage of time, water requirement is expected to increase, with additional burden being passed on to the groundwater sources, compounding the problem of contamination of groundwater due to solid waste disposal in the landfill.

4.7 Impact of Bhalaswa Landfill on Groundwater

Analysis of the model results was carried out to determine the impact of model parameters viz. time period of simulation, equivalent height of leachate, depth on the transport of contaminants from Bhalaswa landfill. Simulation of the model was carried out by varying the values of the parameters. Analysis was carried out for the two parameters Chloride, and DOC, for the time period of 5, 10, 15, 20, 25, and 30 years. Equivalent height was taken as 10, 15, and 20m.

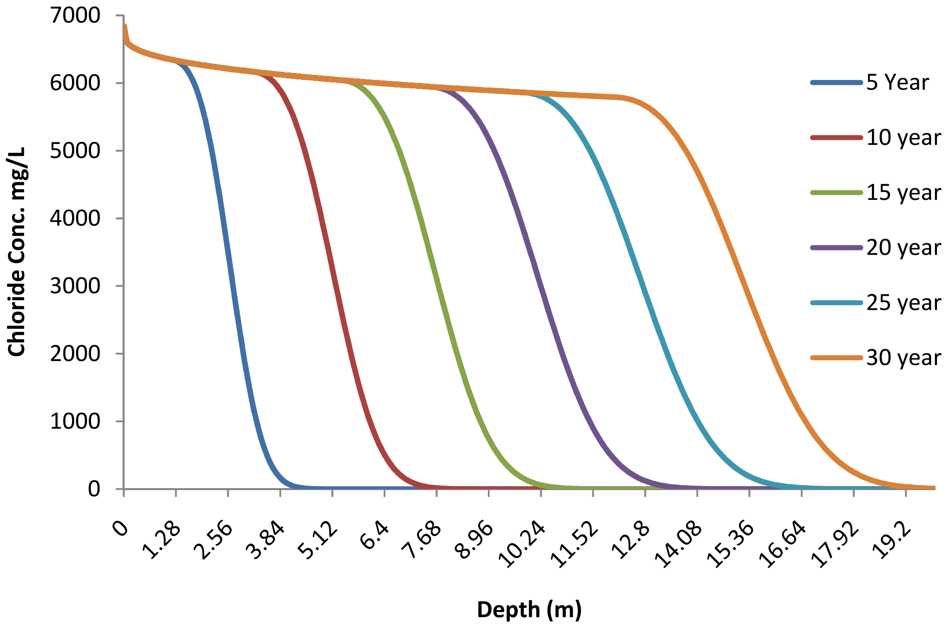


Fig. 4.6 Variation of Chloride Conc. from Bhalaswa Landfill (H_f=10m)

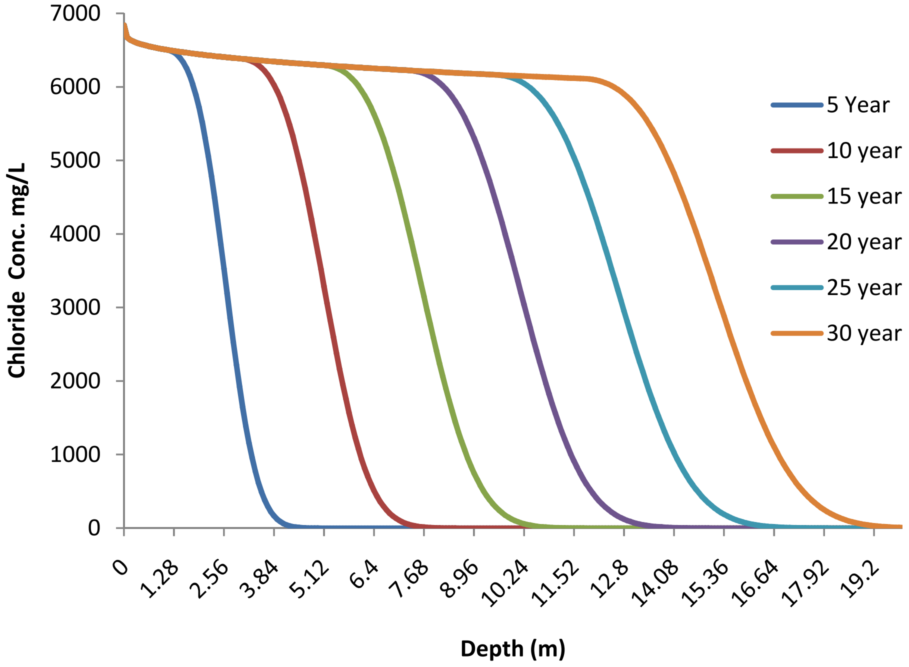


Fig. 4.7 Variation of Chloride Conc. from Bhalaswa Landfill (Hf=15m)

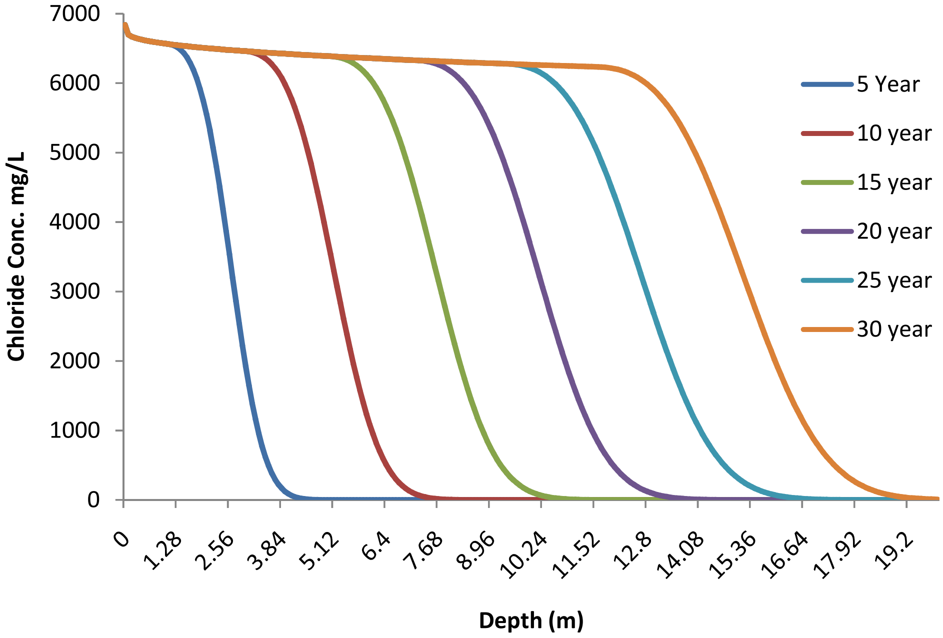


Fig. 4.8 Variation of Chloride Conc. from Bhalaswa Landfill (H_f=20m)

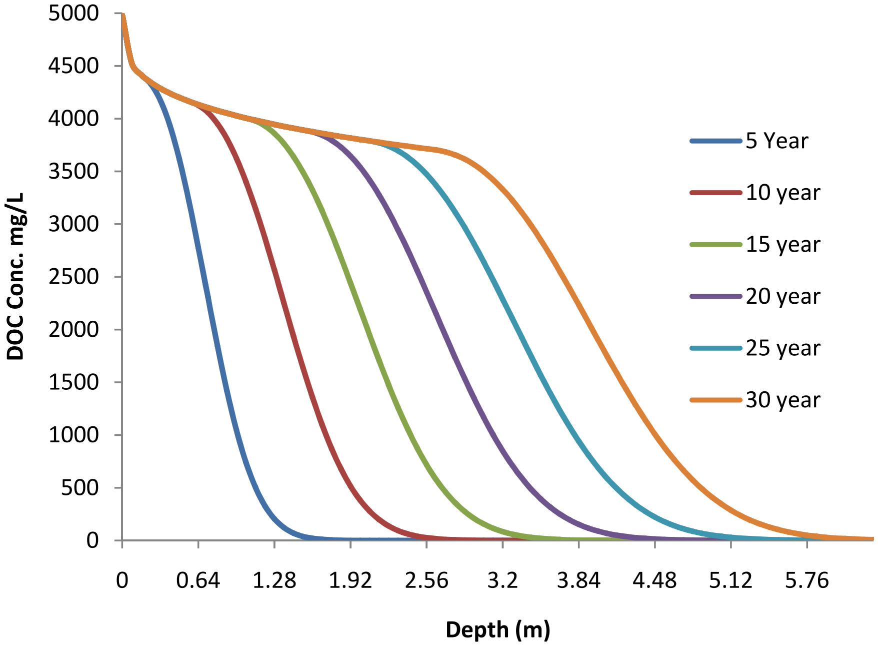


Fig. 4.9 Variation of DOC Conc. from Bhalaswa Landfill (H_f=10m)

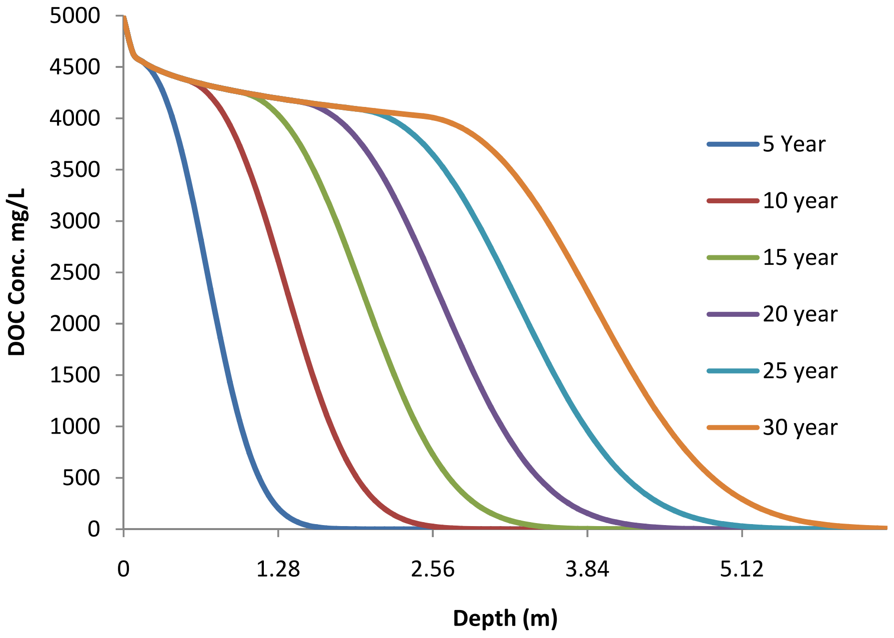


Fig. 4.10 Variation of DOC Conc. from Bhalaswa Landfill (H_f=15m)

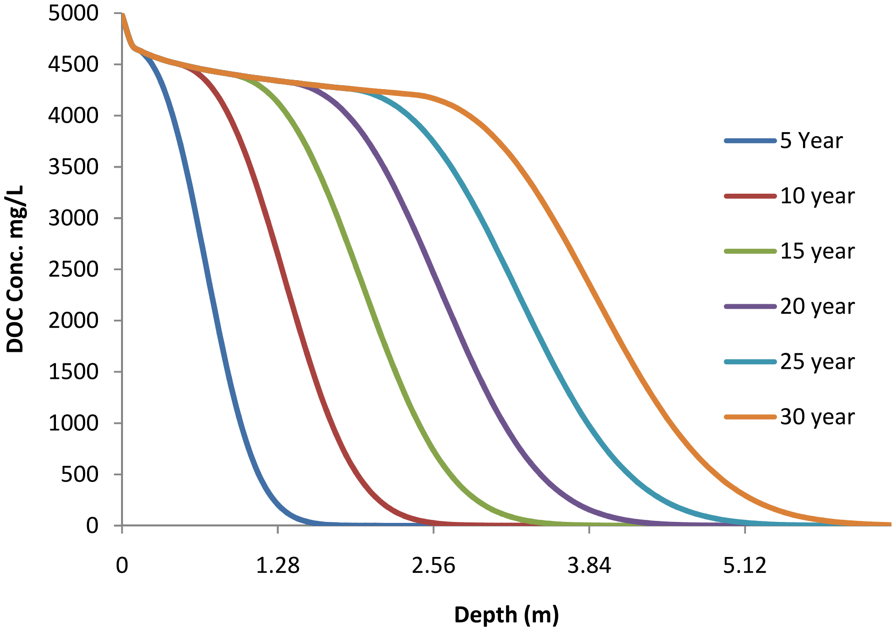


Fig. 4.11 Variation of DOC Conc. from Bhalaswa Landfill (H_f=20m)

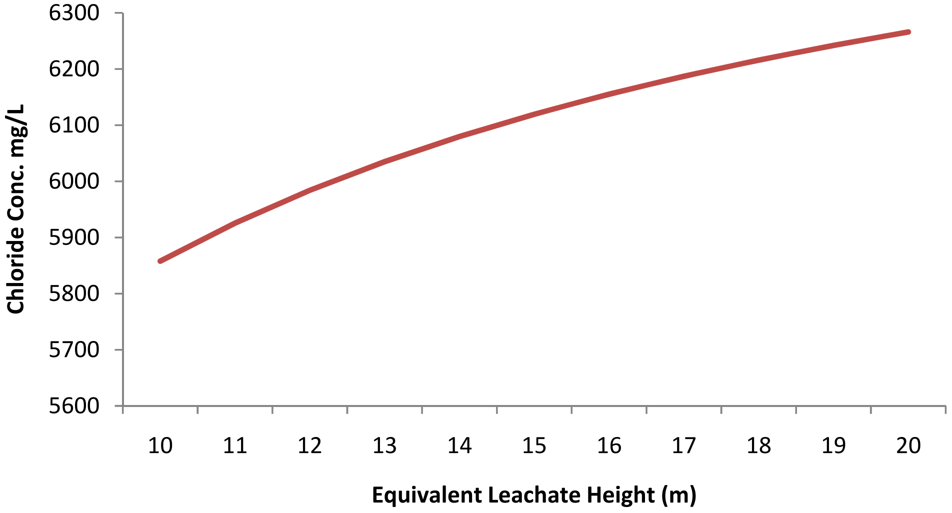


Fig. 4.12 Variation of Chloride Conc. from Bhalaswa Landfill with Respect to Leachate Height at Depth=10m (Time =30 years)

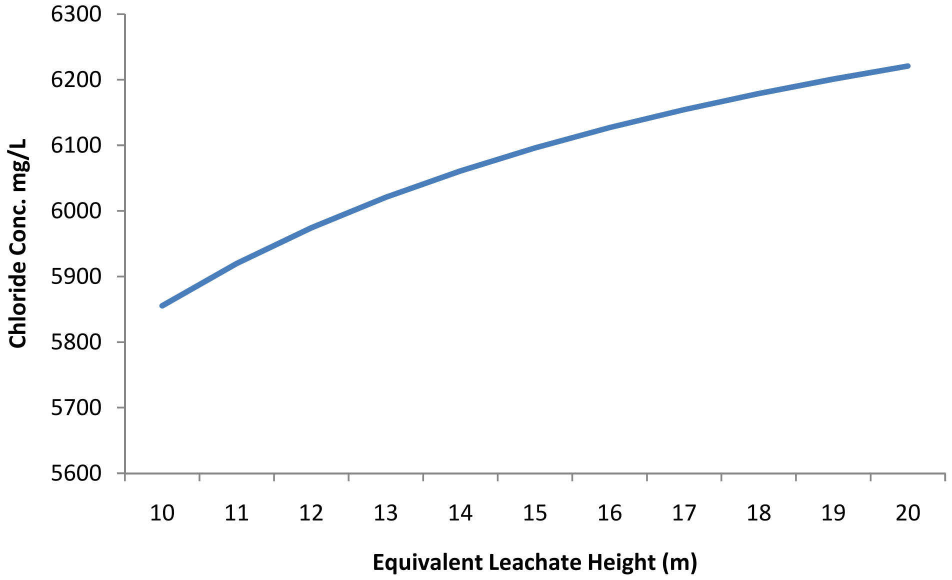


Fig. 4.13 Variation of Chloride Conc. from Bhalaswa Landfill with Respect to Lachate Height at Depth=10m (Time 25 years)

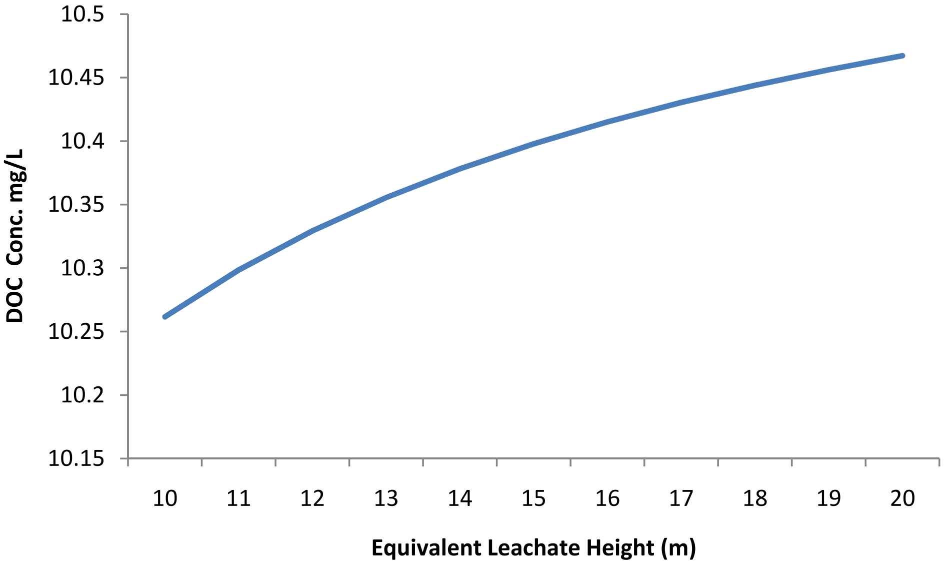


Fig. 4.14 Variation of DOC Conc. from Bhalaswa Landfill with Respect to Leachate Height at Depth=5m (Time =25 years)

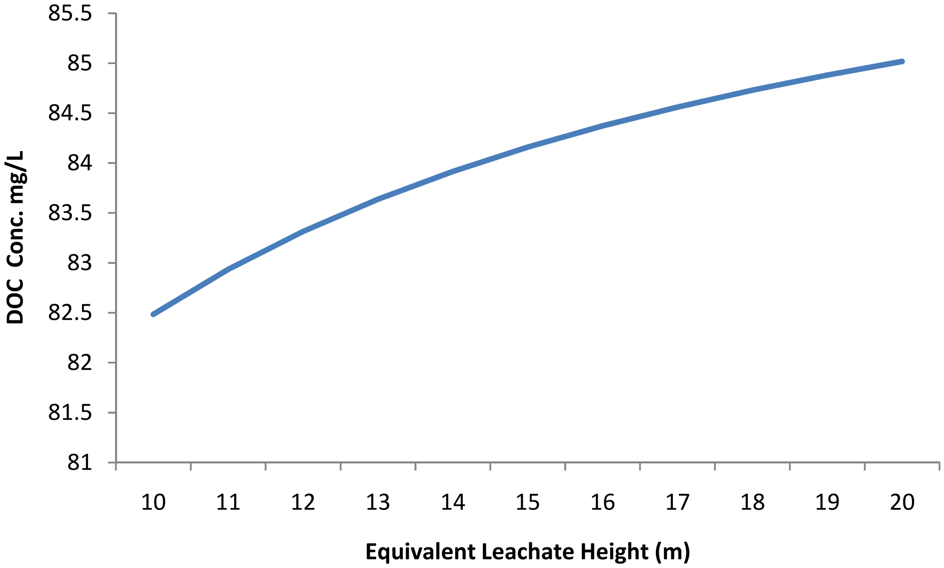


Fig. 4.15 Variation of DOC Conc. from Bhalaswa Landfill with Respect to Leachate Height at Depth=5m (Time =30 years)

From the simulation results obtained and as shown in Fig. 4.6 to Fog. 4.11, it can be seen that the Equivalent height of the leachate will have only marginal effect on the transport of chloride or DOC from Bhalaswa Landfill. That is to say that even if the landfill is continued to be operated for some more, it is not likely to high a high impact on the maximum concentration of the contaminants below the landfill site over a long period of time. Fig. 4.12 and Fig. 4.13 shows the variation in Chloride Concentration at 10m below the landfill with respect to equivalent height of leachate for 30, and 25 years respectively. It can be seen from these figures that increase in leachate height from 10 to 20m results in increase in chloride concentration by about 6% only. Similarly referring to from Fig. 4.14 and Fig. 4.15, it can be seen that the DOC concentration at 5 below landfill increases by about 3%. It can be thus concluded that by hastening or prolonging the closure of the landfill is likely to have only a little effect on the concentration of the contaminants in the groundwater below the landfill.

4.8 Gas Production from Bhalaswa Landfill

The landfill site at Bhalaswa in Delhi receives the wastes from adjoining areas, and a large portion of it is received from nearby vegetable markets. Thus the waste composition received at the landfill site contains a high amount of putrescible organic matter with high moisture content. Such a composition of waste is likely to result in generation of large amount of landfill gas. Presently the site does not have any gas collection or flaring facility, and it is likely to pose a hazard in future as gas generation rates rises.

The expected gas generation from Bhalaswa Landfill site were computed using LandGem software. LandGEM is based on a first-order decomposition rate equation for quantifying emissions from the decomposition of landfilled waste in municipal solid waste (MSW) landfills. The software provides a relatively simple approach to estimating landfill gas emissions. Model defaults are based on empirical data from U.S. landfills. Field test data can also be used in place of model defaults when available. LandGEM is considered a screening tool, the better the input data, the better the estimates. Often, there are limitations with the available data regarding waste quantity and composition, variation in design and operating practices over time, and changes occurring over time that impact the emissions potential. Changes to landfill operation, such as operating under wet conditions through leachate recirculation or other liquid additions, will result in generating more gas at a faster rate.

The historical solid wastes deposition data at Bhalaswa landfill site are not available. However, the latest trends show the deposition rate of about 2500-3000 tonnes per day of solid wastes. Thus, starting from the year of beginning of the landfill site 1992, and assuming a growth rate of solid waste being received at the site to be around 7%, the total amount of the solid waste deposited so far was arrived at and used for the computations of gas generation using LandGem software.

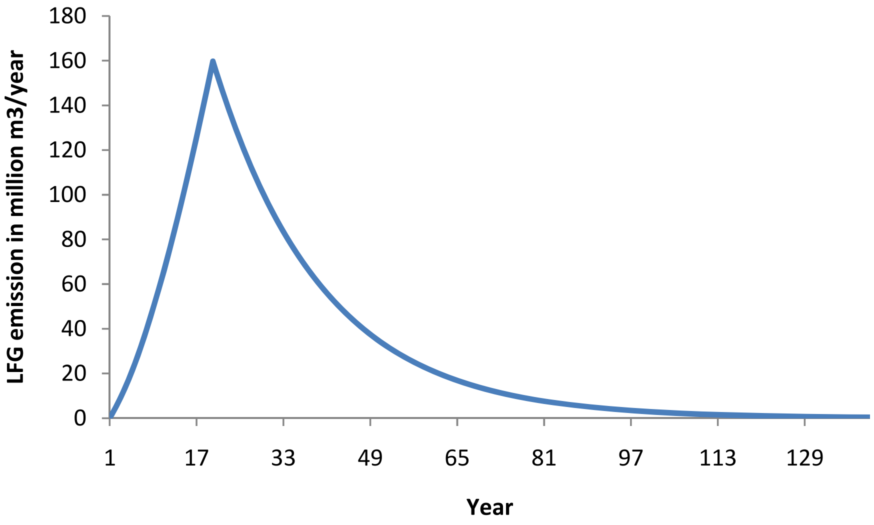


Fig. 4.16 Total LFG Emissions from Bhalaswa Landfill

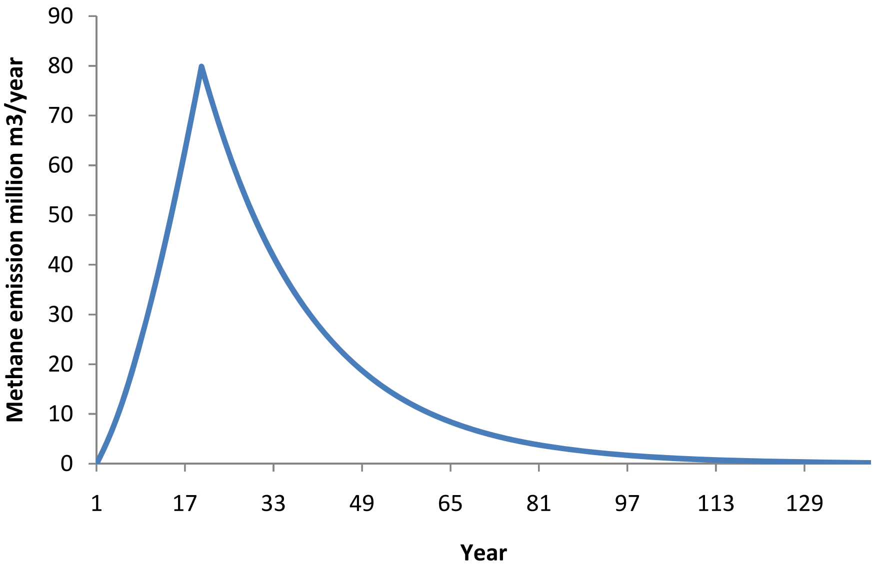


Fig. 4.17 Methane Emission from Bhalaswa Landfill

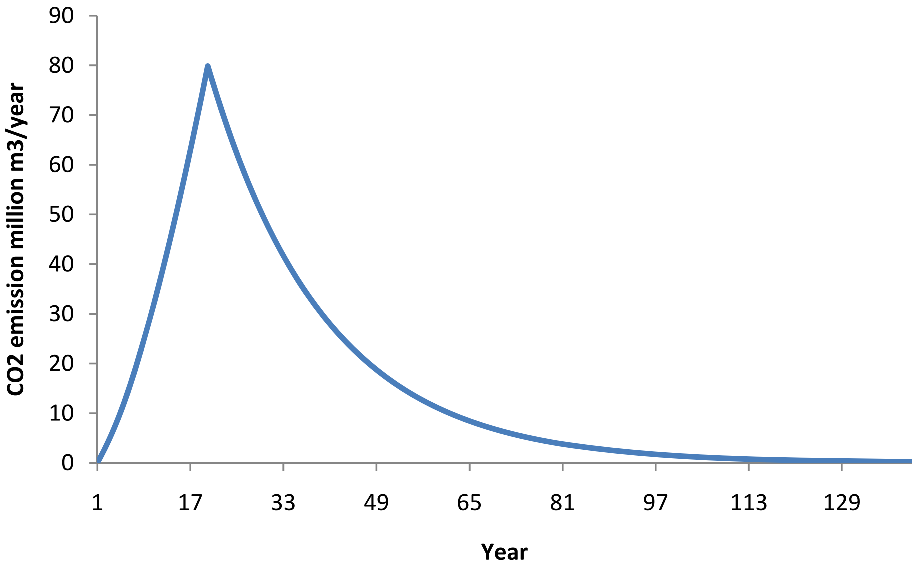


Fig. 4.18 CO_2 Emission from Bhalaswa Landfill

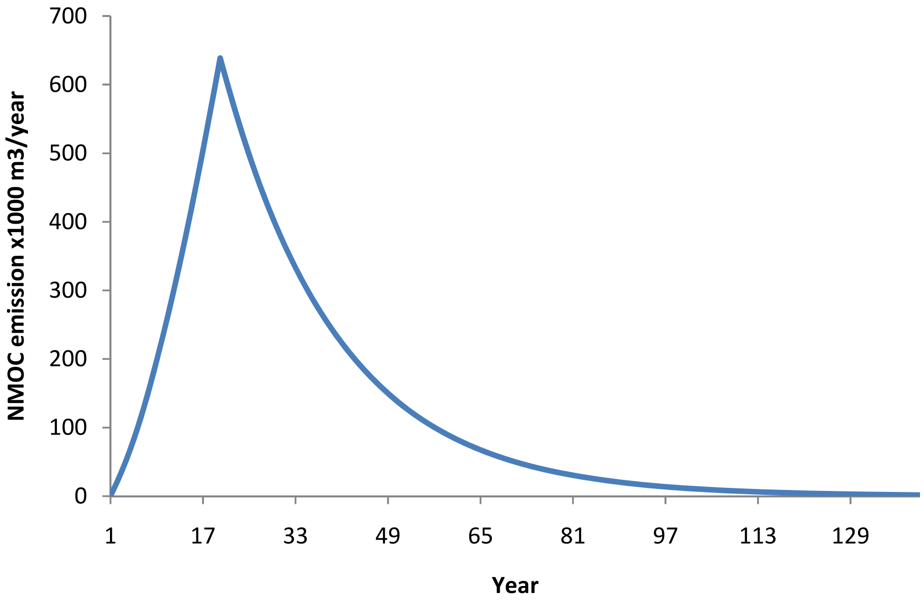


Fig. 4.19 NMOC Emission from Bhalaswa Landfill

From Fig. 4.17 it can be seen that the methane emission rate follows a triangular pattern reaching a maximum value of about 80 million m^3/year after about 25 years of landfill closure. The total methane emission was calculated on the basis of area of the Fig. 4.17, and was found that the on an average the methane emission rate from Bhalaswa landfill stands at about 250 m^3/m^2/year.

Chapter 5

Design Charts for Landfill Liner Thickness

CHAPTER 5

DESIGN CHARTS FOR LINER THICKNESS

5.1 Introduction

The key requirement of a sustainable landfill is prevention of harm to the environment potentially achieved in number of ways, which requires effective landfill design, engineering and management, so that long term environmental risks are acceptable. The containment systems for modern sanitary landfills involve the use of barrier layers to prevent leachate from leaving the landfill and contaminating the underlying soil and groundwater, to prevent water from entering the landfill to create leachate and to trap, divert and collect the gases evolved within the body of landfill. The containment layer at the bottom of a landfill is known as a liner and the one at the top is referred to as a cap or cover. Liners provide the final line of defense against groundwater contamination. The protection of groundwater from migrating chemical species from landfill leachate can only be achieved by providing a suitable landfill liner having an adequate thickness, and the same needs be designed on rational basis.

5.2 Minimum Thickness of Landfill Liner

The model developed in this study was used for preparation of design charts which can be used to determine the minimum thickness of landfill liner to satisfy the pollution prevention criteria of a region. For the preparation of such design charts, simulations of the model were run for a range permeability values ranging from 0.03 cm/yr to 3 cm/yr for compacted clayey liners, and 6 cm/yr to 10 cm/yr for aquitards (Munro et al. 1997). The design charts have been prepared for a range of equivalent height of leachate ranging from 0.1m to 10m to represent the mass of landfilled waste for conservative contaminant species from landfill like Chloride, Zinc, etc. and for a range of retardation factors varying from 1 to 100 for Nonconservative contaminant species like DOC. A conservative contaminant is considered one which does not show attenuation either due to sorption or due to decay/reaction.

The equivalent leachate height representing the density of waste in a landfill has been defined earlier. If the percentage of contaminant that is present in the waste is known through chemical analysis or through typical landfill data charts, and the total design weight of the landfill facility is available, then the total mass of contaminant can be determined. The maximum concentration of the contaminant species of interest in leachate can either be determined by analysis or from the typical landfill data. Knowing the maximum concentration of contaminant species of interest in leachate and the corresponding leachable mass of that contaminant species along with the aerial extent of the landfill, the equivalent height of leachate can be determined (Rowe et al. 1995).

For the purpose of design charts the simulation of the model was run with finite mass of the contaminant as the upper boundary, where the initial concentration of contaminant in leachate was taken as 1000 mg/L and the porosity of barrier/aquitard was assumed as 0.4. The model gives the spatial and temporal variation of concentration along the thickness of the barrier and the maximum concentration recorded in each depth for the entire domain (50 Years). The simulation was run for time domain of 50 years, and the time step was determined after satisfying the stability criteria of solution. The time step was thus in the range of 0.04 year to 0.25 year for the element sizes ranging from 0.06m to 0.08m. Initial concentration (background) was assumed to be zero for a fully flushed boundary. The conservative contaminant front in all the cases could not reach the bottom boundary for the cases of both, small and large permeability values. The maximum concentration attained at various depths for a number of leachate heights was determined next. The procedure was followed for every equivalent leachate height and permeability value considered. The parameters taken for the preparation of these design charts are listed in Table 5.1. It has been observed that the molecular diffusion coefficient for the conservative or non conservative contaminant has a very small range and is generally taken as 0.02 m^2/yr (Rowe 1988). Similarly the dispersivity of clay ranges from 0.01 to 0.14 m depending upon the texture of clayey soil (Munro et al. 1997) and this range has been considered from the preparation of these design charts. Fig. 5.1 to Fig. 5.30 represent the design charts for the range of permeability values for the conservative solute (e.g. chloride).

From the results plotted graphically, one can determine the minimum design thickness of landfill liner knowing the permissible concentration of contaminant species of interest in groundwater. The permissible value when divided by the typical maximum concentration of that contaminant in the landfill waste gives the design maximum relative concentration, and the appropriate minimum thickness of landfill liner for that designated leachate height can be determined from these figures. On comparing figures, one observes that there is hardly any variation in the respective design charts for the permeability values of 0.03 cm/yr and 0.3 m/yr respectively. This implies that the advection is not the primary transport mechanisms at such low velocities and that the contaminant transport takes place on account of diffusion only.

As the permeability value increases ten fold, the minimum liner thickness required also increases. In case where there is no leachate collection system, the value of equivalent leachate height will be larger. The maximum relative concentration is determined by computing the ratio of permissible chloride concentration (250 mg/L) to the maximum chloride concentration in the landfill. It can be observed that by doubling the permeability value, at maximum relative concentration of 0.12 and at equivalent leachate height of 10m, the minimum aquitard thickness also increases by three times. Since advection becomes an important transport mechanism at high permeability values, a small increase in the velocity has predominant effect on the aquitard thickness required. However when a leachate collection system is provided at a landfill site, the value of equivalent height of leachate is reduced. For the case when leachate height is equal to 1m (implying that the 90% of the leachate is collected in the leachate collection system located at the bottom of landfill), the amount of leachate that infiltrates the barrier is about 10% only.

Table 5.1 Simulation Data

S.No.	**Parameter**	**Value**
Common Data		
1	Duration	50 years
2	Porosity	0.40
3	Effective Molecular Diffusion Coefficient	0.02 m^2/year
Neoconservative contaminant		
4	Retardation Factor (R_f)	1,3,6,24,100
Conservative contaminant		
5	Equivalent Height of Leachate	10, 5, 1, 0.5, 0.1 m

5.3 Effect of Sorption on Liner Thickness

At low permeability values and for most of the contaminants found in municipal landfills (Rowe et al. 1995), the computed retardation factor assumes a value ranging from 1 to 100. The design charts have been prepared taking into account the retardation factor values of 1, 3, 6, 24, and 100. The charts are shown in Fig. 5.31 to 5.60. From these charts one can determine the minimum thickness of liner based on retardation factor depending upon the extent of attenuation by means of sorption, decay or reaction involved. The retardation factor value may be computed either on the basis of partition coefficient value or can determined by carrying out laboratory tests on the contaminant species of interest. The minimum thickness of liner for the non-conservative contaminant can be computed in the way similar to that of conservative contaminant. Maximum relative concentration needs to be determined on the basis of permissible maximum concentration of the contaminant at the bottom of the landfill liner and the typical maximum concentration of that contaminant species of interest in landfill leachate. As can be observed from the Fig. 5.31 to 5.60, due to attenuation by sorption or decay the minimum value of the liner thickness reduces.

5.4 Design Liner Thickness

The minimum liner thickness to be provided shaould be computed for the different contaminant species of interest, and the maximum value of all these thickness is expressed as design value of liner thickness and the same is ultimately provided for a landfill liner. In the case of municipal landfill the major contaminants can be considered to be Chloride as conservative contaminant, and Dissolved Organic Carbon (DOC) as non-conservative contaminant. Thus for determining the design thickness the minimum thickness required for each of these contaminant is computed and the larger of the two is provided as design value of liner thickness. Also in municipal landfill leachate, of all the metal contaminants, zinc is generally found to be having largest concentration, and it may be subjected to attenuation or not depending upon the actual filed conditions. For determining the minimum liner thickness based on mass transport of zinc the conditions would have to be assessed and partition coefficient may be determined if needed. The minimum thickness for landfill liner may then be determined by referring to design charts applicable.

5.5 Design Method for Liner Thickness

The method to be followed for the determination of liner thickness using these charts is as under:

a. Fix up the design maximum concentration of contaminant species of interest in landfill leachate. This can be done by collection and analysis of landfill leachate, or by referring to the literature for published data for that region. (A mg/L)

b. The maximum permissible concentration of the contaminant species of interest in the groundwater can be obtained by referring to the literature published by regulatory agencies. (B mg/L)

c. The maximum relative concentration to fulfill the regulatory requirement can be determined simply be getting the ratio of B/A.

d. The equivalent height of leachate can be determined by maximum concentration of the contaminant species of interest in leachate and the mass of that contaminant species in landfilled waste. For example, if a landfill leachate is known to have a maximum concentration of a contaminant to be 4000 mg/L, and the mass of that contaminant species, either determined by laboratory tests or from published design data is 4×10^{10} mg, the aerial extent of landfill being 10^6 sq.m., the equivalent height of leachate shall be $4 \times 10^{10}/(4000 * 10^6) = 10$ m.

e. The hydraulic conductivity or permeability value of the landfill liner can either be determined by carrying out laboratory tests or by referring to the literature on permeability of liner materials.

f. Knowing the maximum relative concentration, and permeability, one can refer to the design chart of interest, for the particular equivalent height of leachate. The corresponding depth for the maximum relative concentration of interest shall be the minimum thickness of liner required.

5.6 Design Example

Population to be served	5,00,000
Per capita waste generation	0.5 Kg/capita/day
Period of landfill operation	30 yr
Total mass the waste deposited	2737500000 Kg
Area of the landfill	250000 sq.m.
Chloride content in landfill mass (Rowe et al. 1995)	0.2%
Total weight leachable of chloride from landfill	0.002*2237500000
	= 5475000000 mg
Chloride concentration in landfill leachate	= 1000 mg/L
Equivalent leachate height	$= \frac{5475000000}{(5000*250000)}$
	= 4.38 m, Say 5 m
Permissible concentration of chloride	= 250 mg/L
Design maximum relative concentration	= 250/1000
	= 0.25

Permeability of the barrier = 0.03 cm/yr

Referring to Fig. 5.3, the depth corresponding to maximum relative concentration of 0.25 is 1440 mm. Thus minimum thickness of liner for such condition can be fixed up as 1.44m.

Fig. 5.1 Variation of Maximum Relative Concentration
[Coservative contaminant]Permeability = 0.03 cm/yr

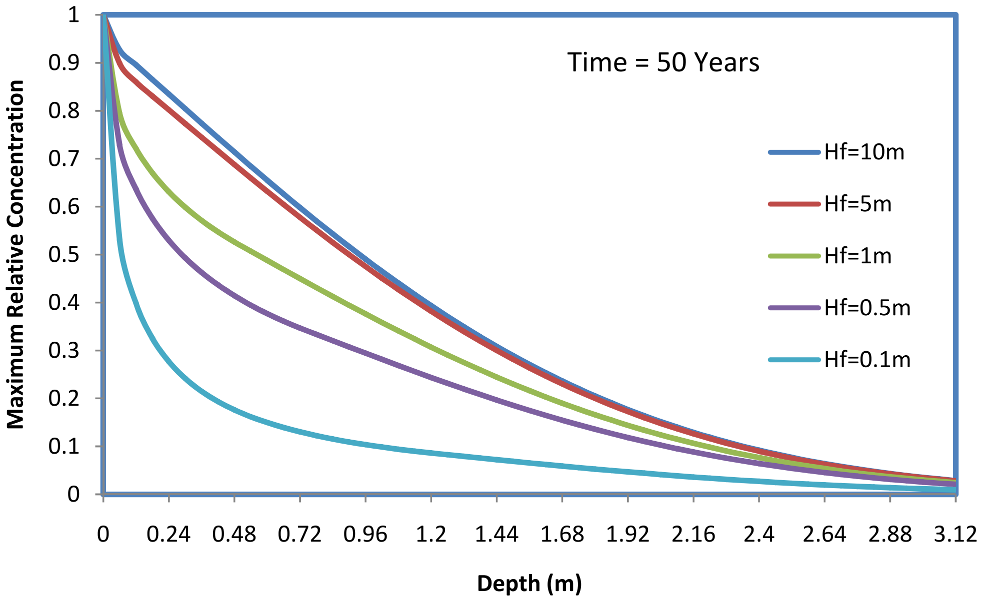


Fig. 5.2 Variation of Maximum Relative Concentration [H_f=10m]
[Coservative contaminant]Permeability = 0.03 cm/yr

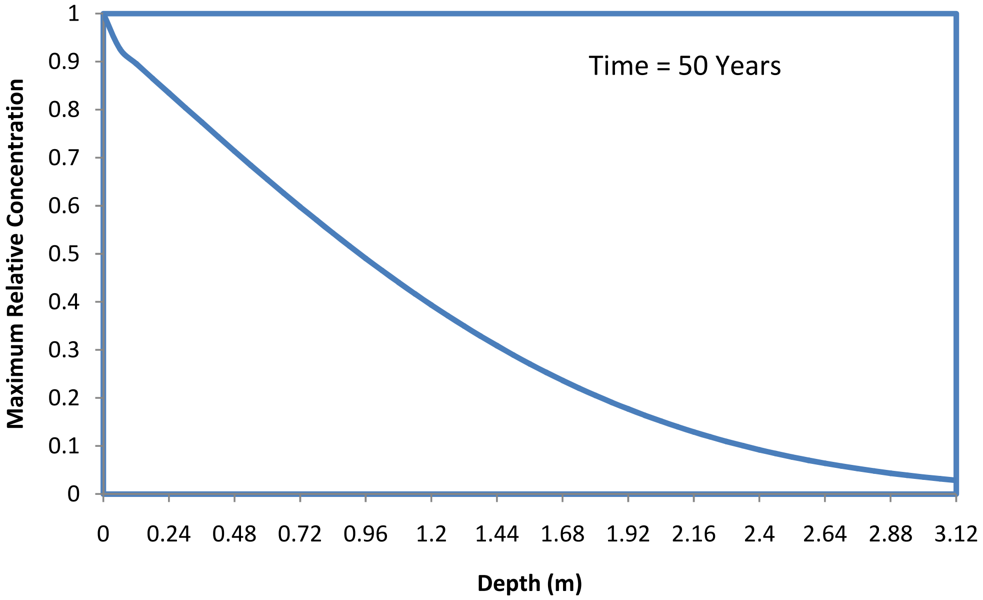


Fig. 5.3 Variation of Maximum Relative Concentration [H_f=5m] [Coservative contaminant]Permeability = 0.03 cm/yr

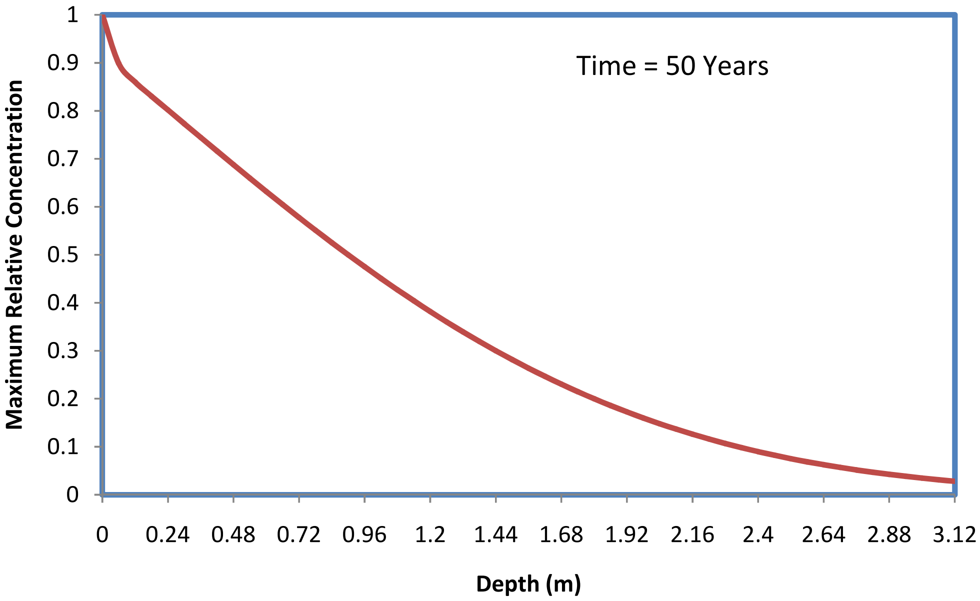


Fig. 5.4 Variation of Maximum Relative Concentration [H_f=1m] [Coservative contaminant]Permeability = 0.03 cm/yr

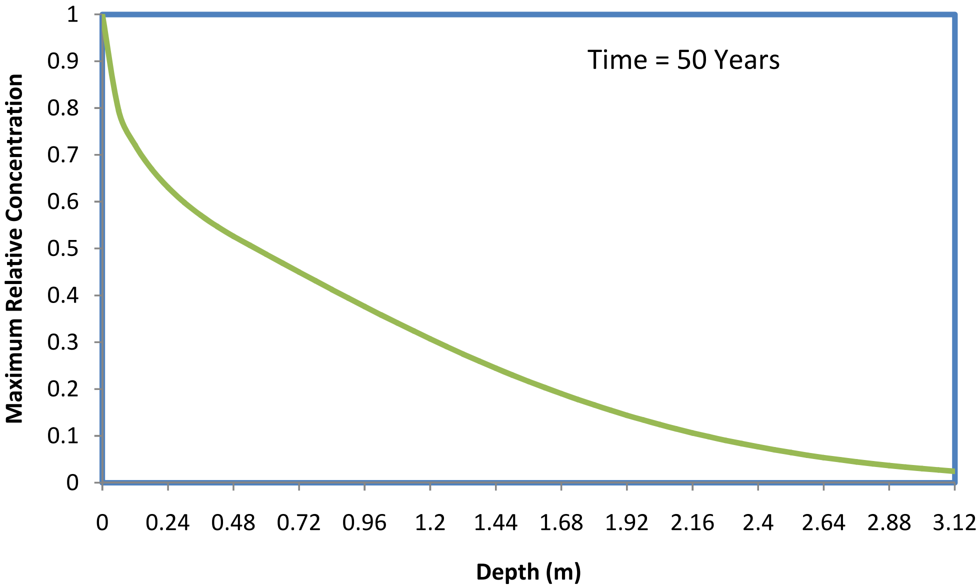


Fig. 5.5 Variation of Maximum Relative Concentration [H_f=0.5m]
[Coservative contaminant]Permeability = 0.03 cm/yr

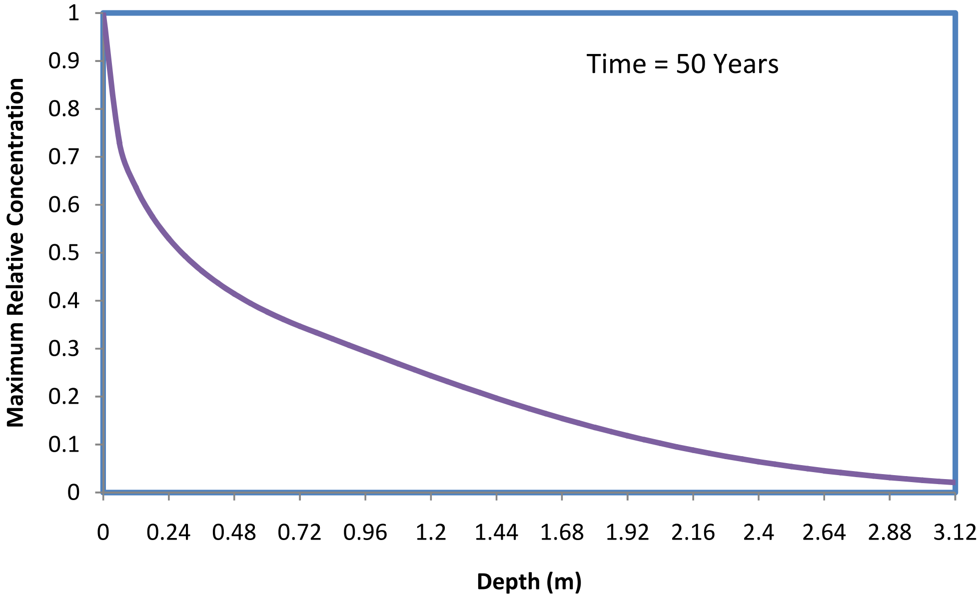


Fig. 5.6 Variation of Maximum Relative Concentration [H_f=0.1m]
[Coservative contaminant]Permeability = 0.03 cm/yr

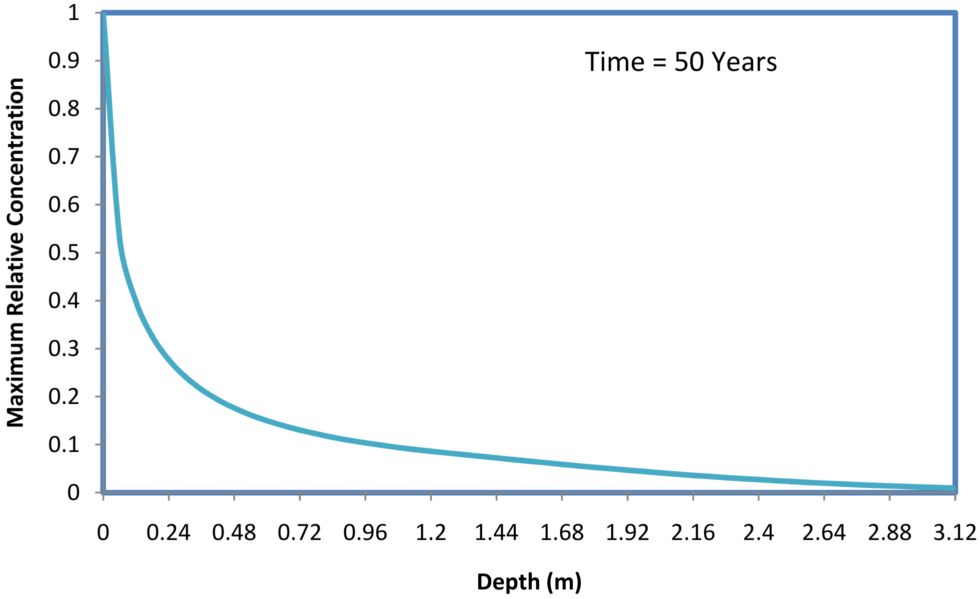


Fig. 5.7 Variation of Maximum Relative Concentration [Coservative contaminant]Permeability = 0.3 cm/yr

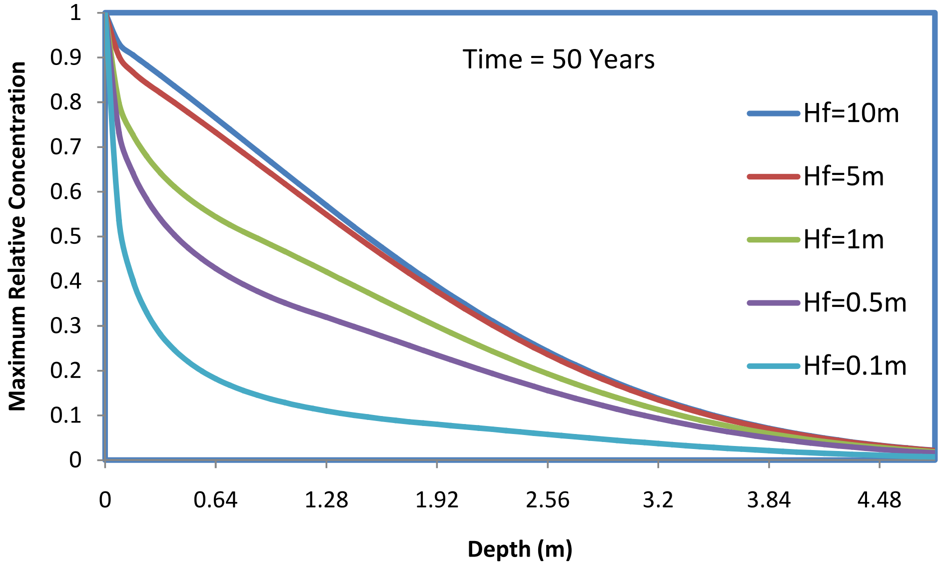


Fig. 5.8 Variation of Maximum Relative Concentration [H_f=10m] [Coservative contaminant] Permeability = 0.3 cm/yr

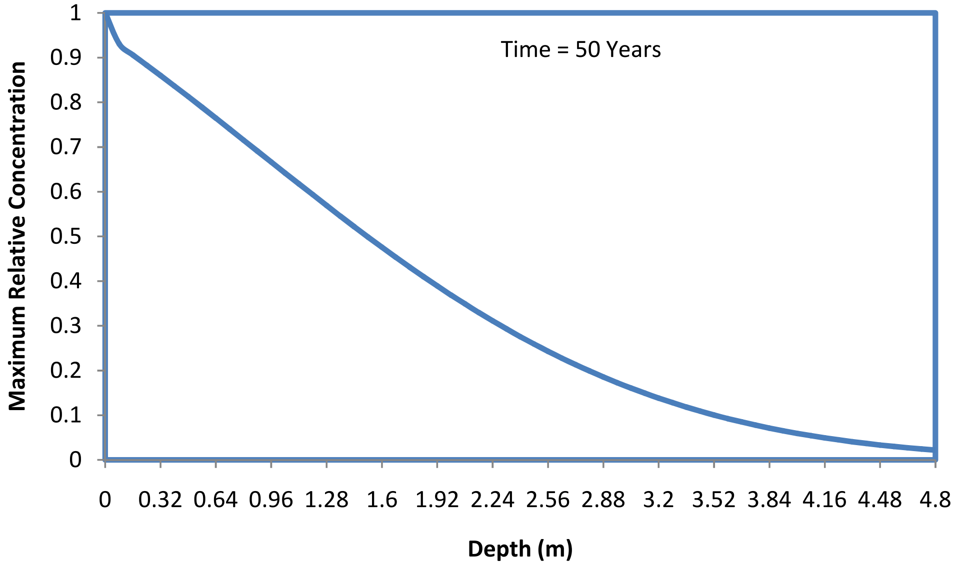


Fig. 5.9 Variation of Maximum Relative Concentration [H_f=5m] [Coservative contaminant] Permeability = 0.3 cm/yr

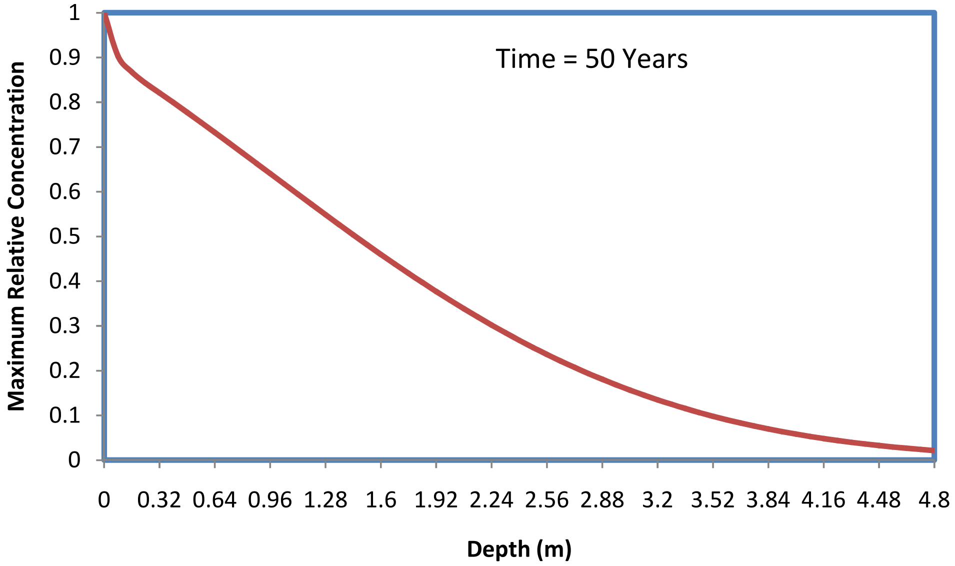


Fig. 5.10 Variation of Maximum Relative Concentration [H_f=1m] [Coservative contaminant] Permeability = 0.3 cm/yr

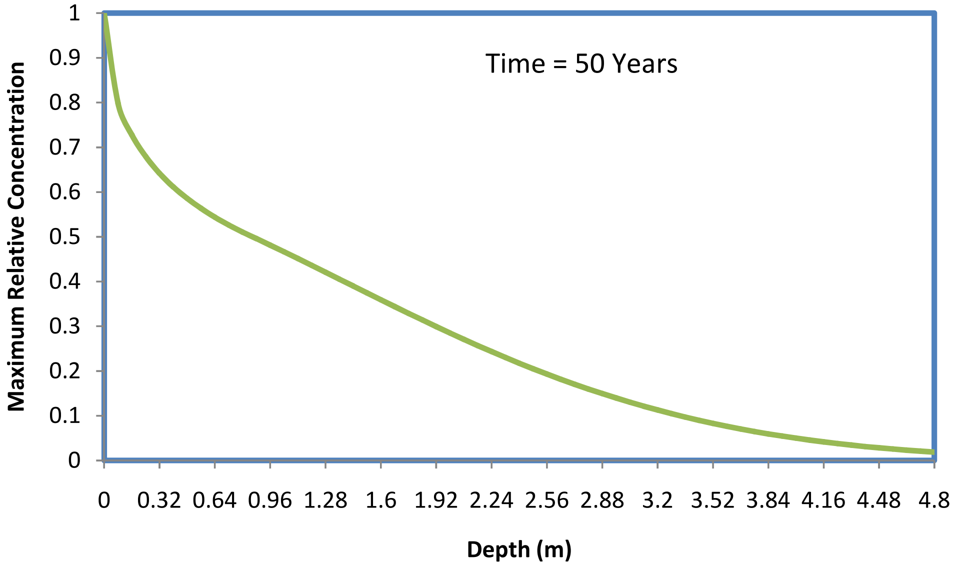


Fig. 5.11 Variation of Maximum Relative Concentration[H_f=0.5m] [Coservative contaminant] Permeability = 0.3 cm/yr

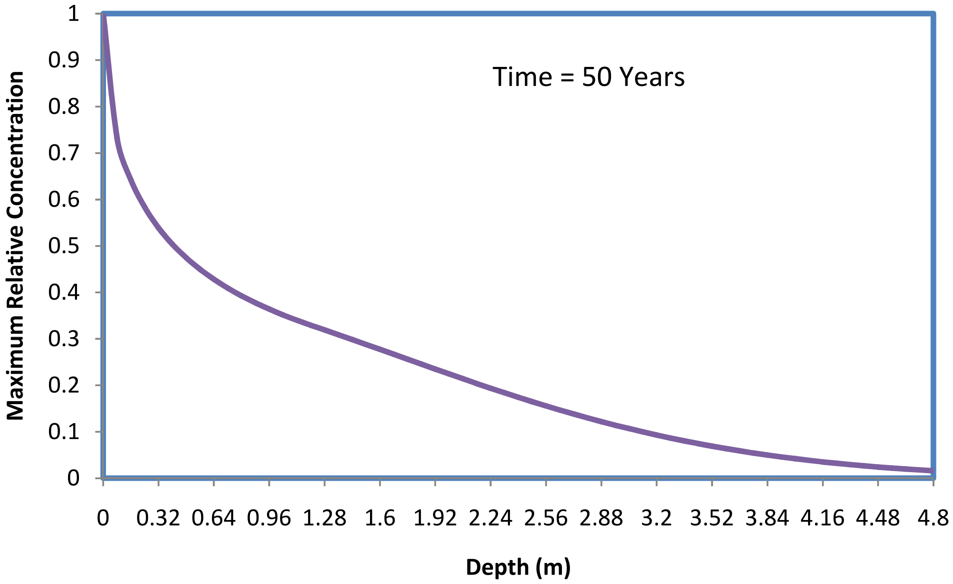


Fig. 5.12 Variation of Maximum Relative Concentration[H_f=0.1m] [Coservative contaminant] Permeability = 0.3 cm/yr

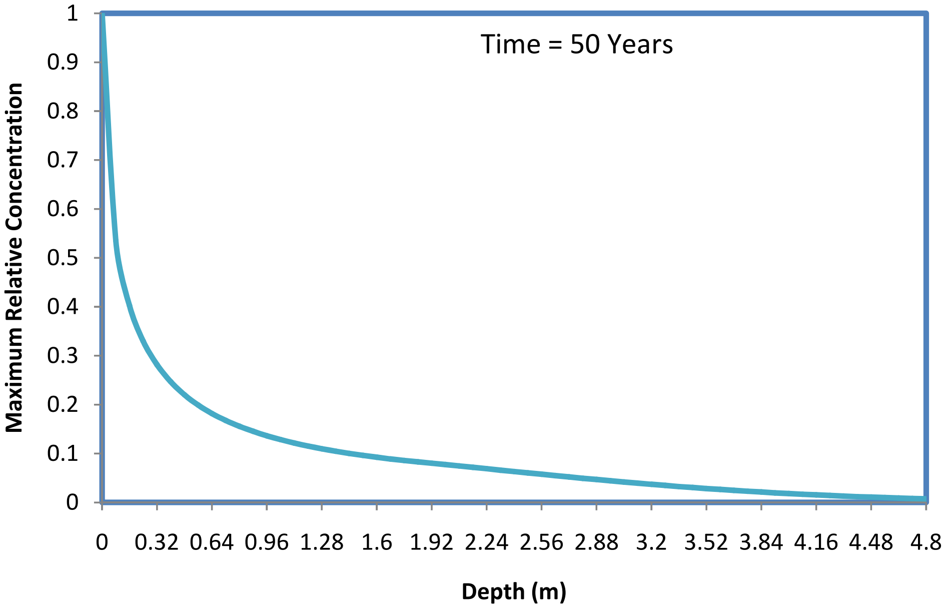


Fig. 5.13 Variation of Maximum Relative Concentration [Coservative contaminant] Permeability=3 cm/yr.

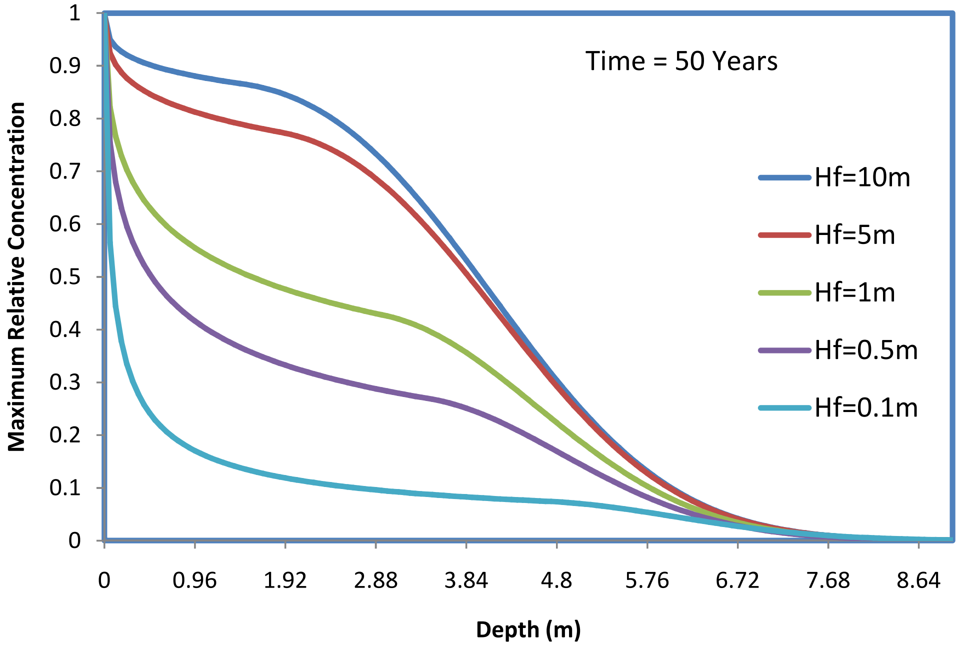


Fig. 5.14 Variation of Maximum Relative Concentration [H_f=10m] [Coservative contaminant] Permeability=3 cm/yr.

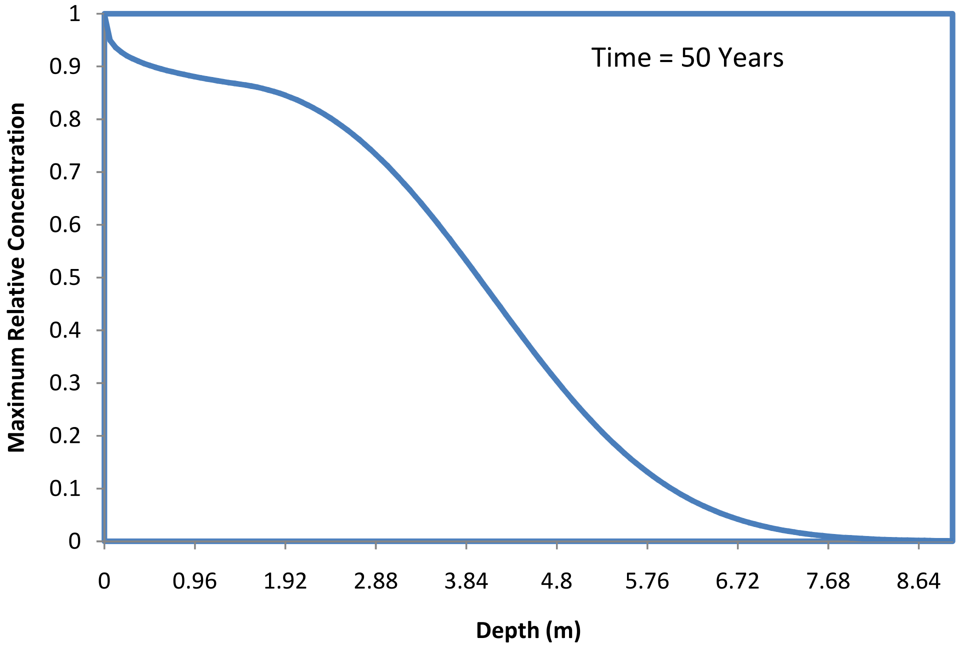


Fig. 5.15 Variation of Maximum Relative Concentration [H_f=5m] [Coservative contaminant] Permeability =3 cm/yr.

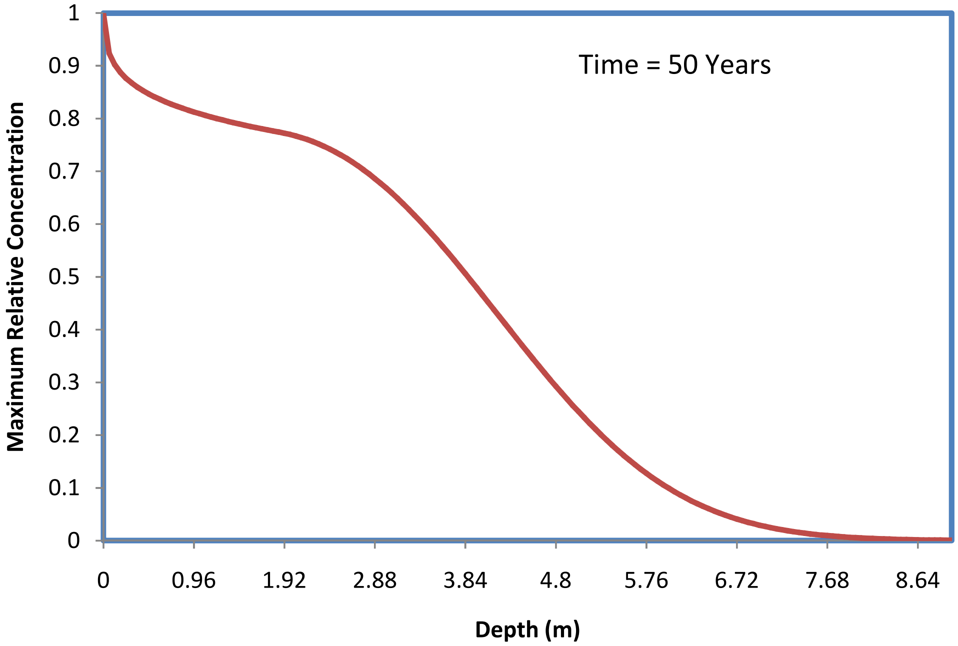


Fig. 5.16 Variation of Maximum Relative Concentration [H_f=1m] [Coservative contaminant] Permeability=3 cm/yr

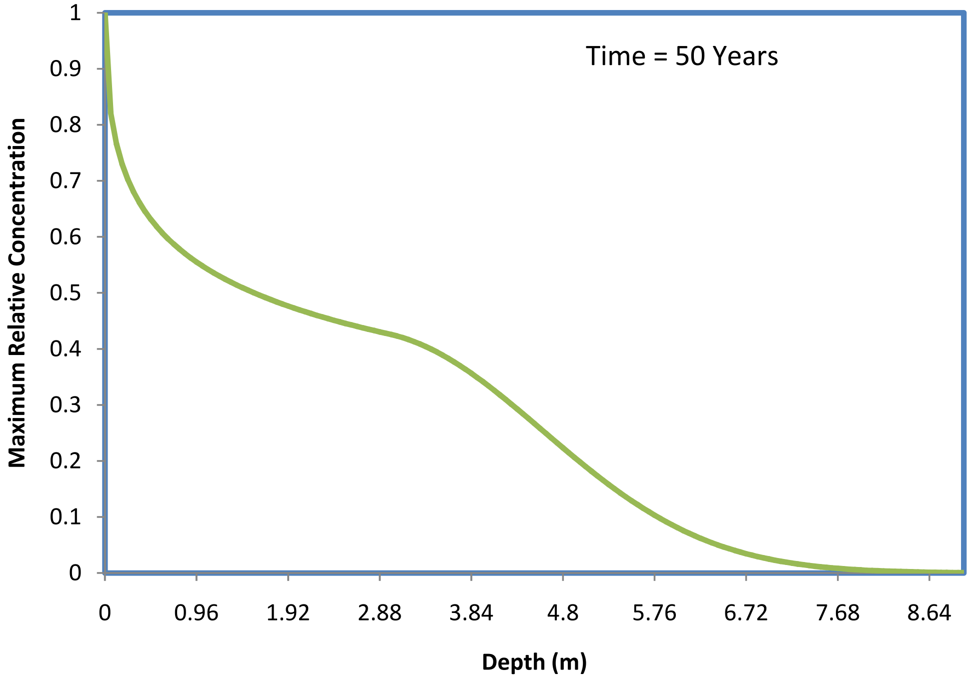


Fig. 5.17 Variation of Maximum Relative Concentration[H_f=0.5m] [Coservative contaminant] Permeability=3 cm/yr.

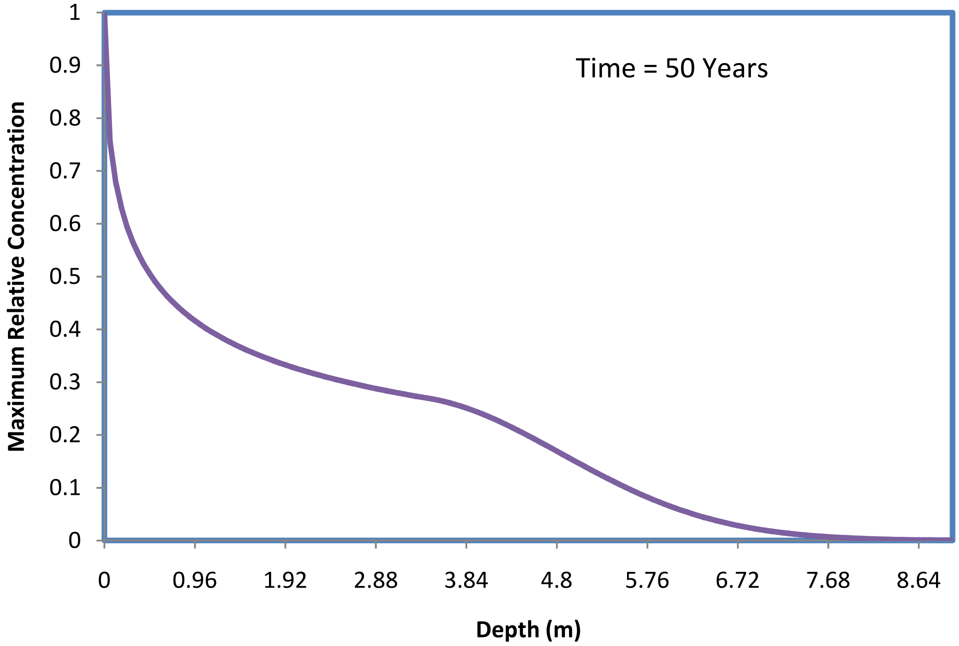


Fig. 5.18 Variation of Maximum Relative Concentration[H_f=0.1m] [Coservative contaminant] Permeability=3 cm/yr.

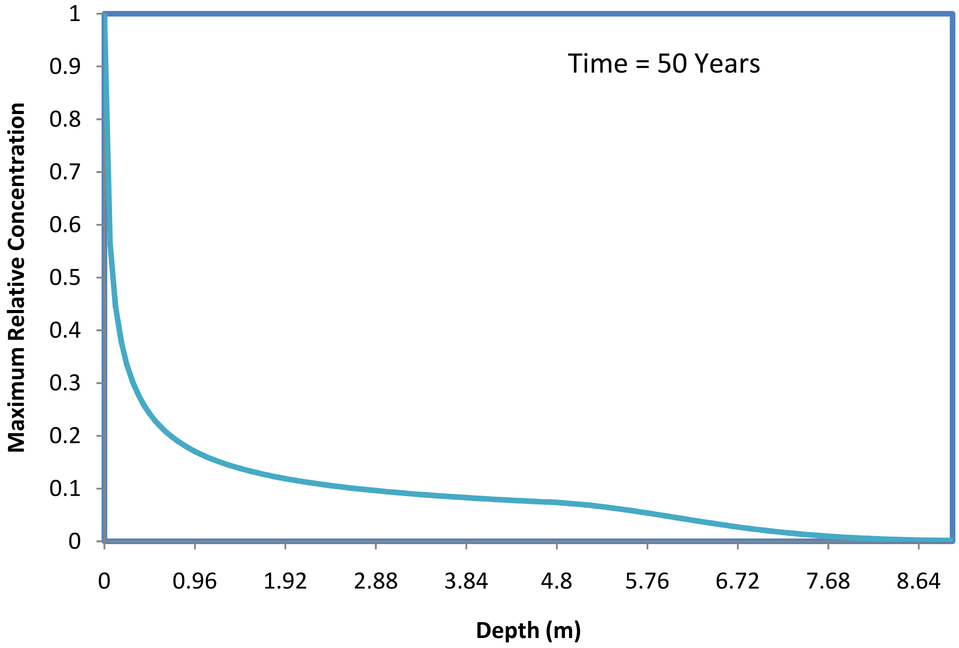


Fig. 5.19 Variation of Maximum Relative Concentration [Coservative contaminant] Permeability= 6 cm/yr.

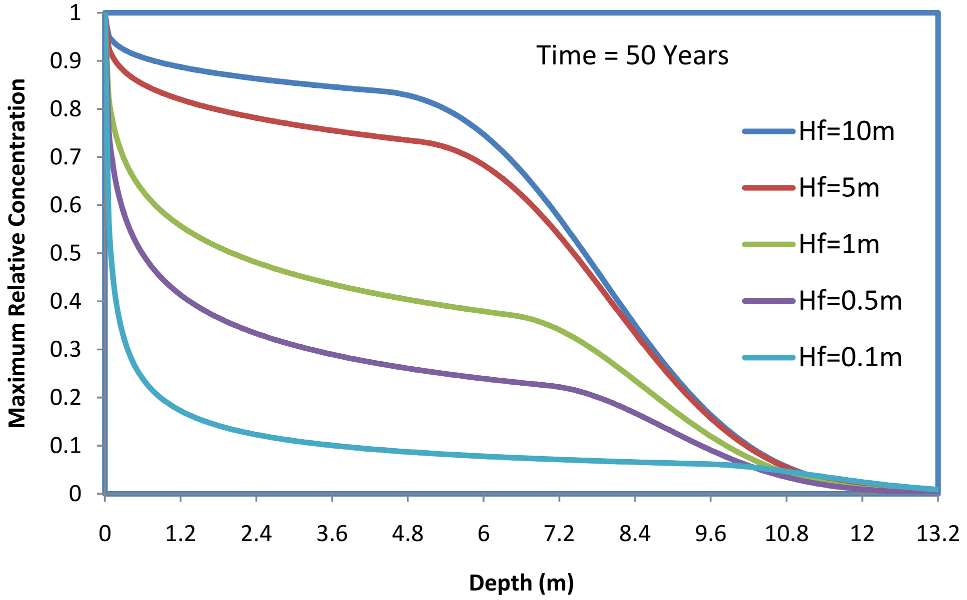


Fig. 5.20 Variation of Maximum Relative Concentration [H_f=10m] [Conservative contaminant] Permeability= 6 cm/yr.

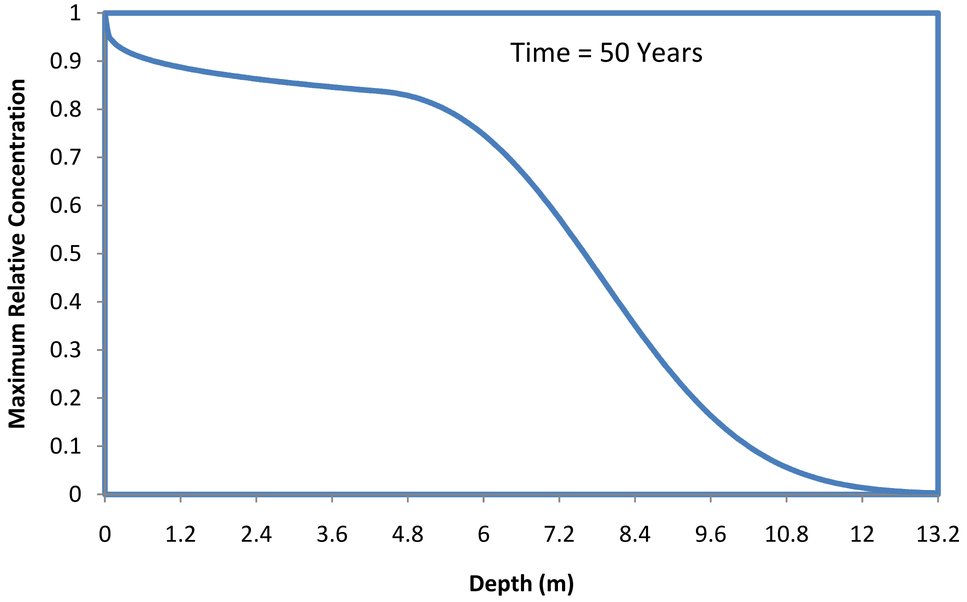


Fig. 5.21 Variation of Maximum Relative Concentration [H_f=5m]
[Conservative contaminant] Permeability= 6 cm/yr.

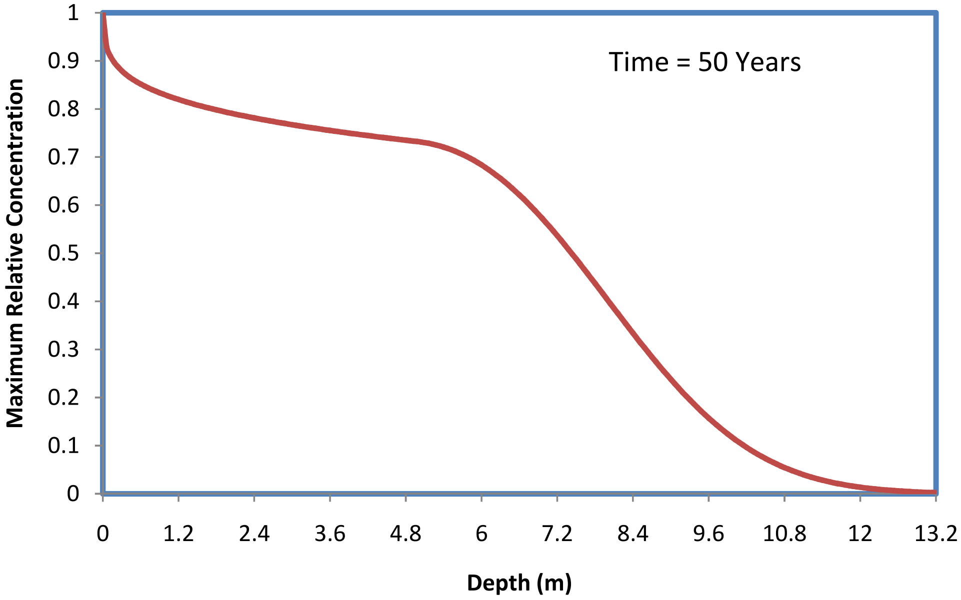


Fig. 5.22 Variation of Maximum Relative Concentration [H_f=1m]
[Coservative contaminant] Permeability= 6 cm/yr.

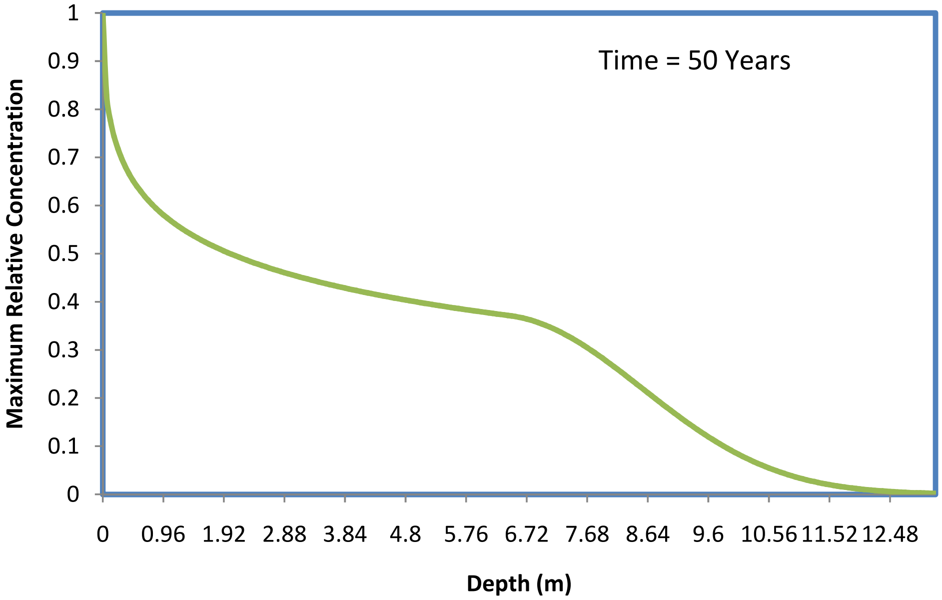


Fig. 5.23 Variation of Maximum Relative Concentration[H_f= 0.5m] [Coservative contaminant] Permeability= 6 cm/yr.

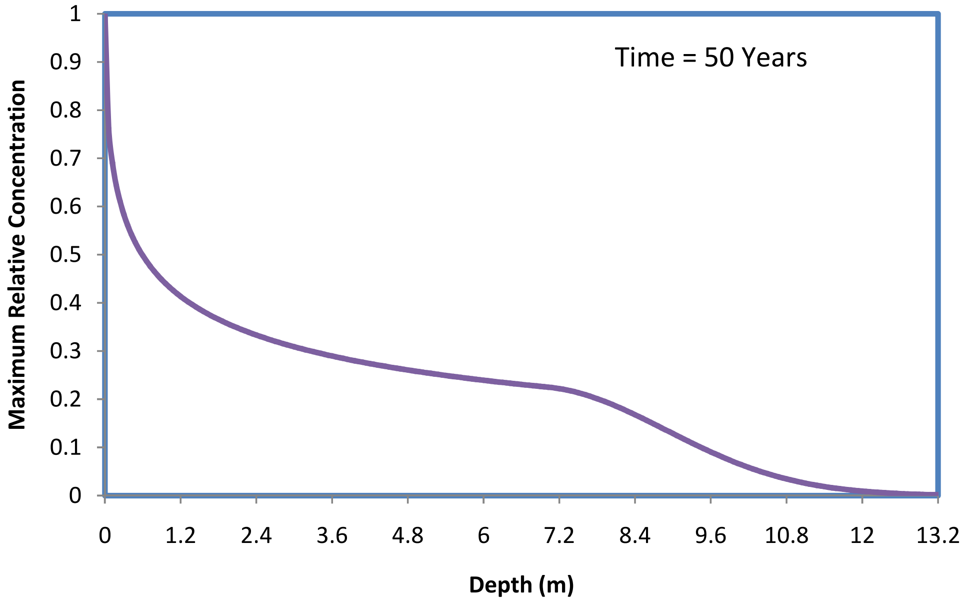


Fig. 5.24 Variation of Maximum Relative Concentration[H_f= 0.1m] [Coservative contaminant] Permeability= 6 cm/yr.

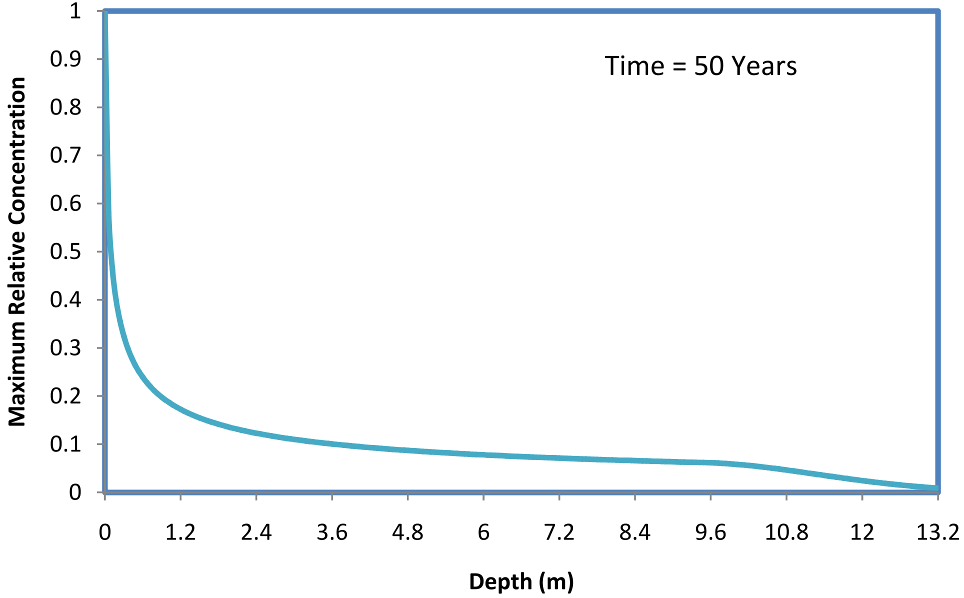


Fig. 5.25 Variation of Maximum Relative Concentration [Coservative contaminant] Permeability = 10 cm/yr.

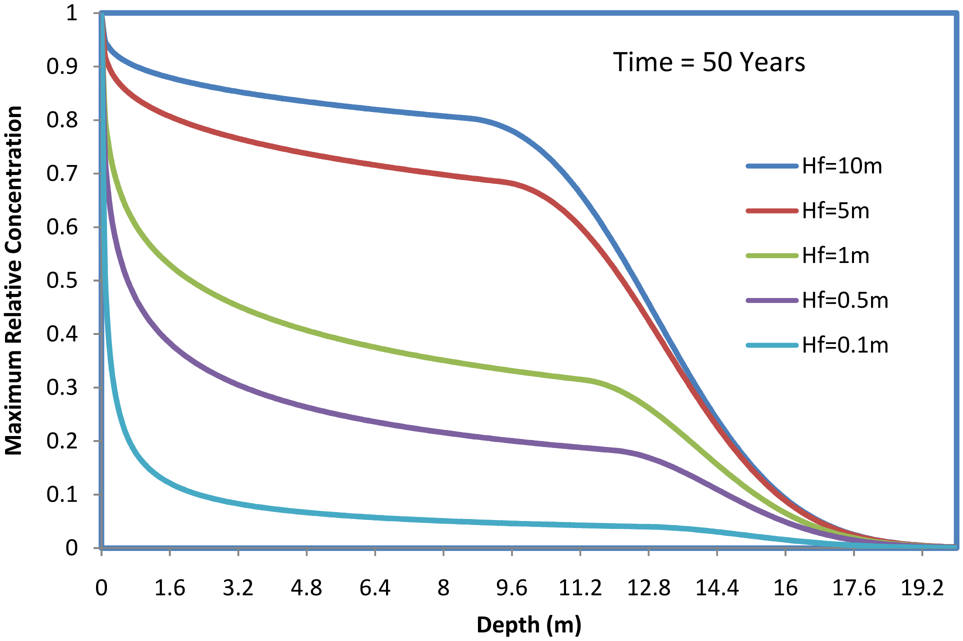


Fig. 5.26 Variation of Maximum Relative Concentration [H_f=10m] [Coservative contaminant] Permeability= 10 cm/yr.

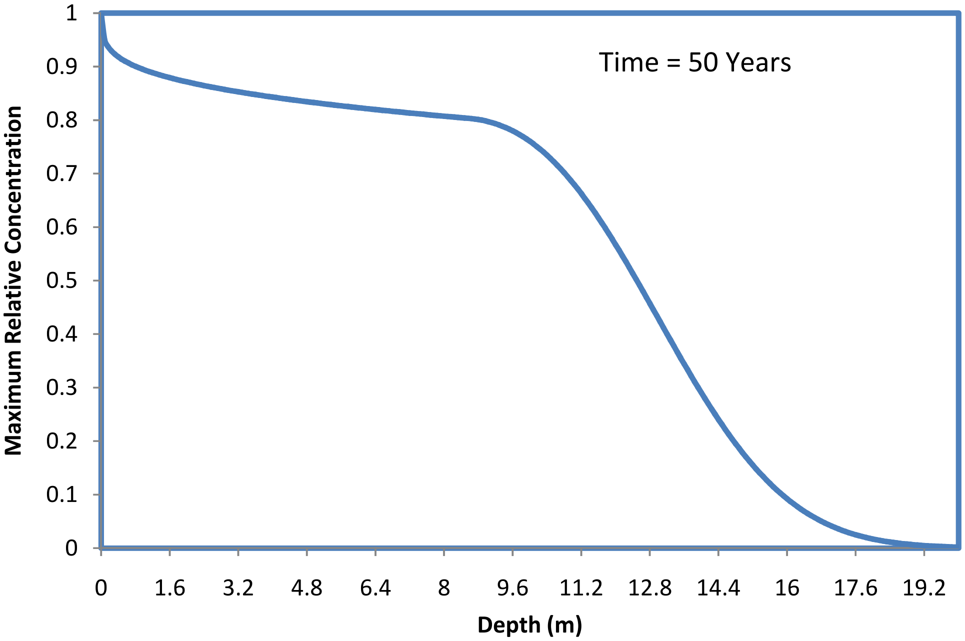


Fig. 5.27 Variation of Maximum Relative Concentration [H_f=5m]
[Coservative contaminant] Permeability= 10 cm/yr.

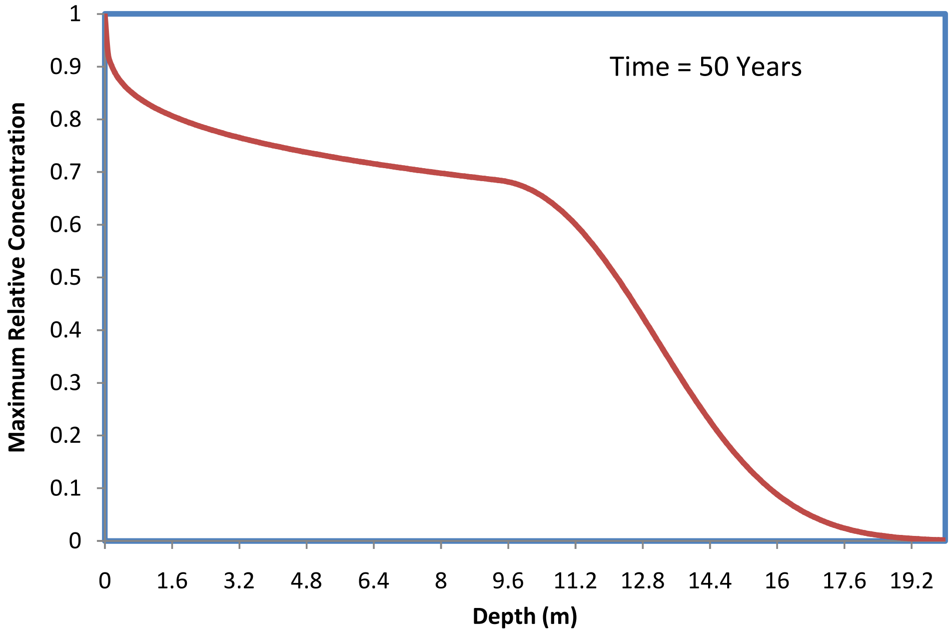


Fig. 5.28 Variation of Maximum Relative Concentration [H_f=1m]
[Coservative contaminant] Permeability= 10 cm/yr.

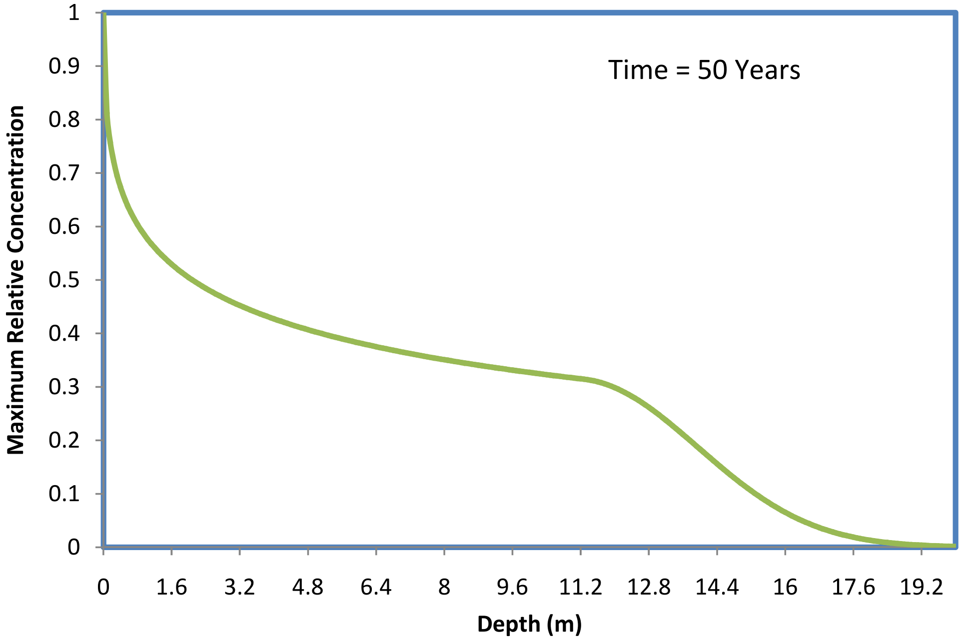


Fig. 5.29 Variation of Maximum Relative Concentration[H_f=0.5m] [Coservative contaminant] Permeability= 10 cm/yr.

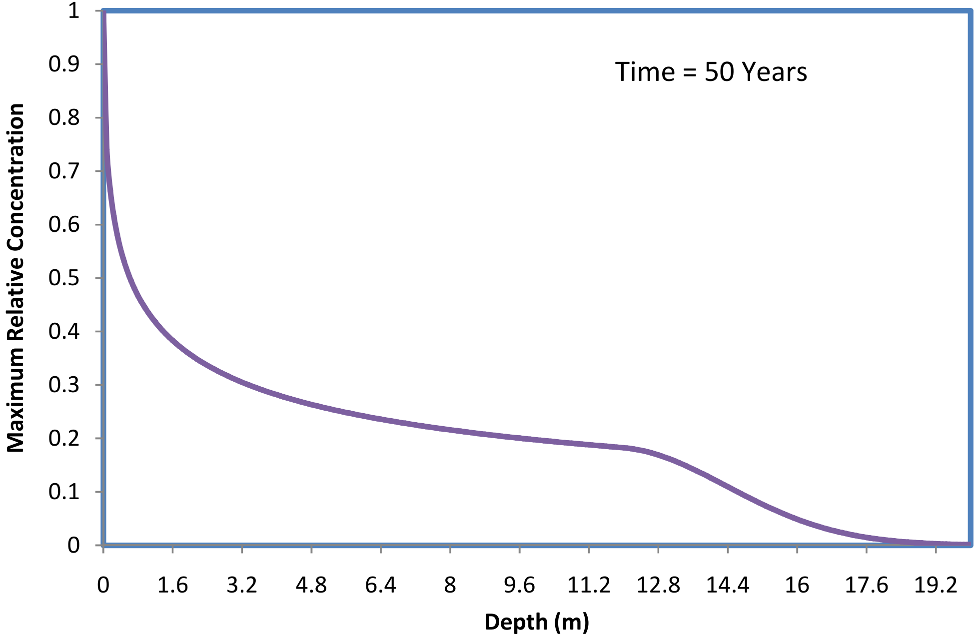


Fig. 5.30 Variation of Maximum Relative Concentration[H_f=0.1m] [Coservative contaminant] Permeability= 10 cm/yr.

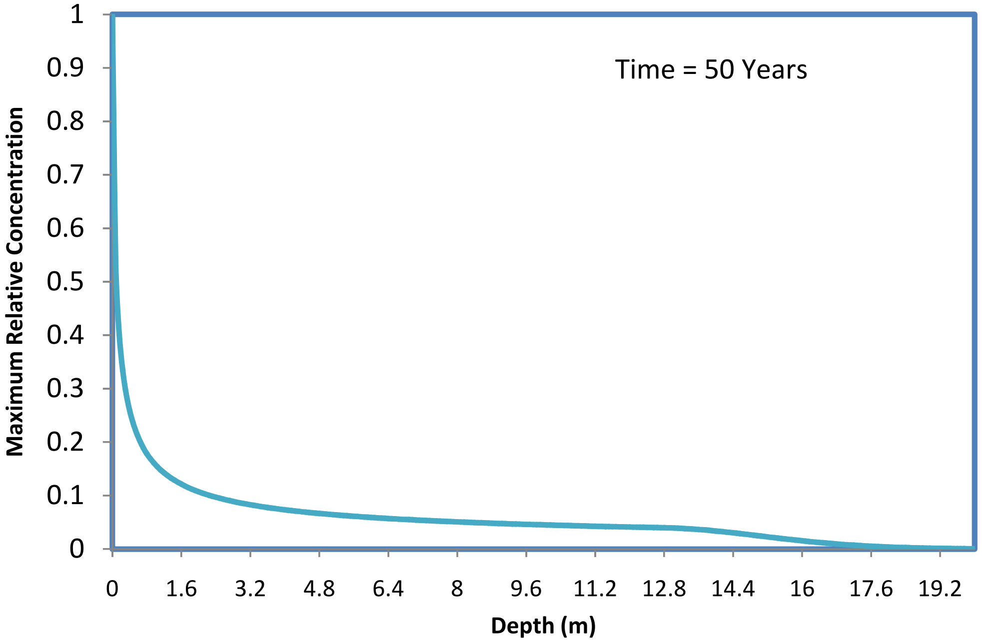


Fig. 5.31 Variation of Maximum relative Concentration [Nonconservative contaminant]Permeability=0.03cm/yr

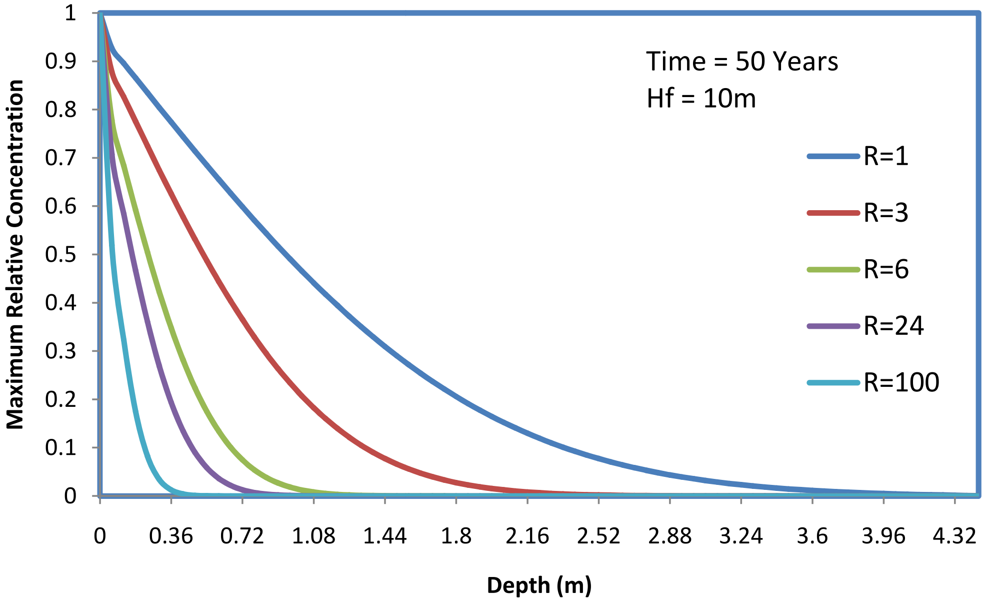


Fig. 5.32 Variation of Maximum relative Concentration [R=1] Nonconservative contaminant]Permeability=0.03cm/yr

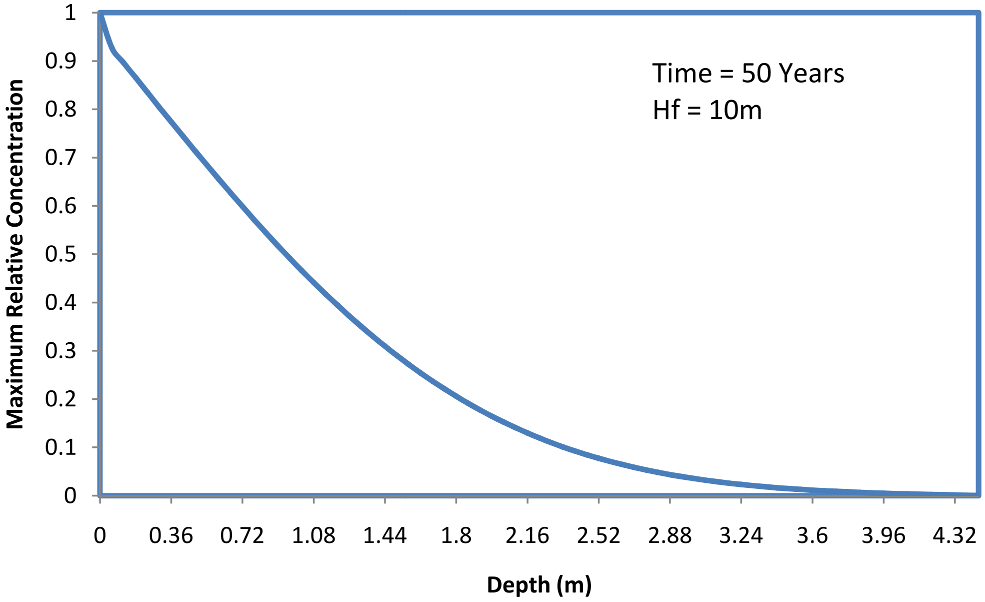


Fig. 5.33 Variation of Maximum relative Concentration [R=3] [Nonconservative contaminant]Permeability=0.03cm/yr

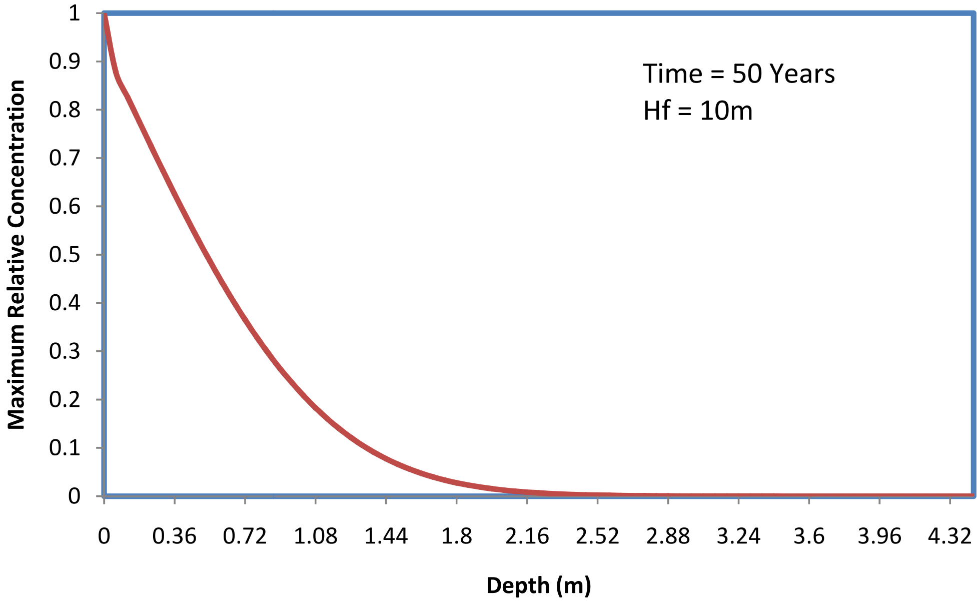


Fig. 5.34 Variation of Maximum relative Concentration [R=6] [Nonconservative contaminant]Permeability=0.03cm/yr

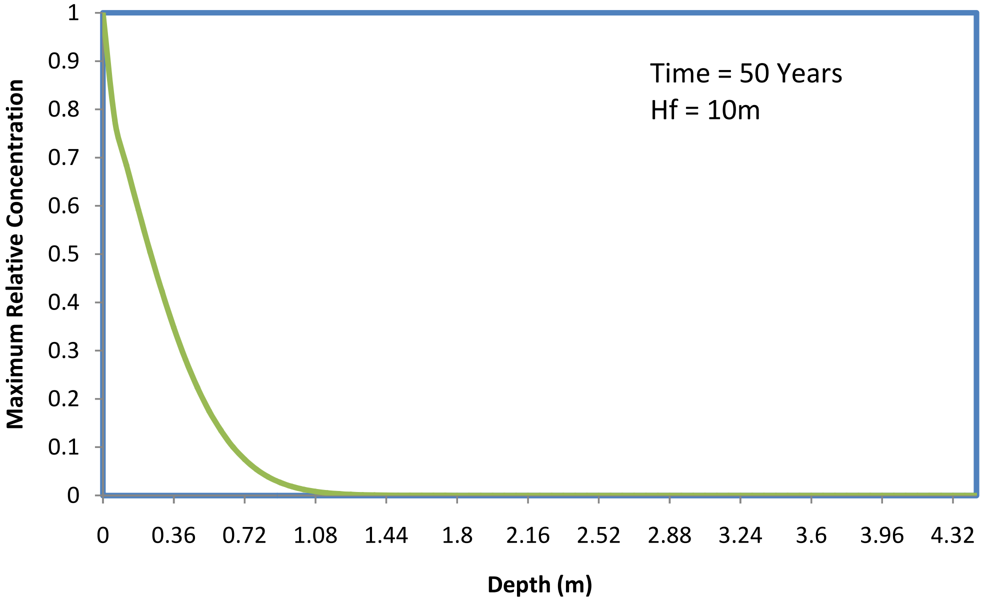


Fig. 5.35 Variation of Maximum relative Concentration [R=24] [Nonconservative contaminant]Permeability=0.03cm/yr

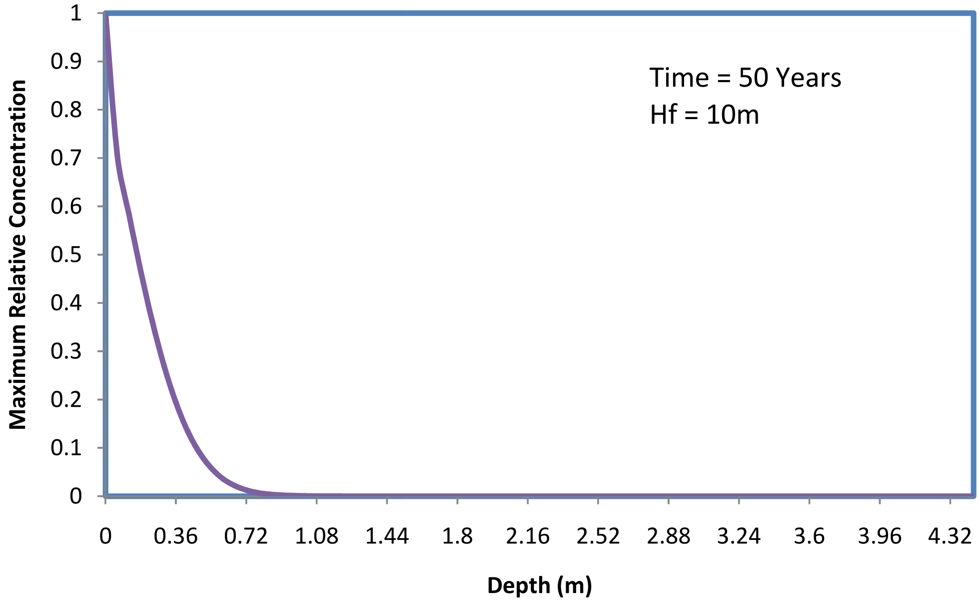


Fig. 5.36 Variation of Maximum relative Concentration [R=100] [Nonconservative contaminant]Permeability=0.03cm/yr

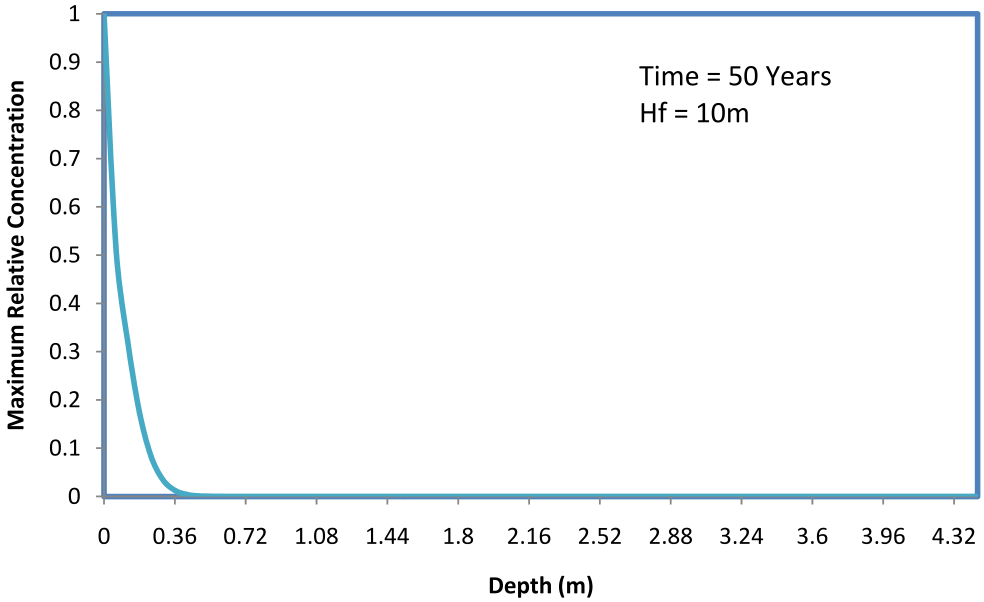


Fig. 5.37 Variation of Maximum Relative Concentration [Nonconservative contaminant]Permeability=0.3cm/yr

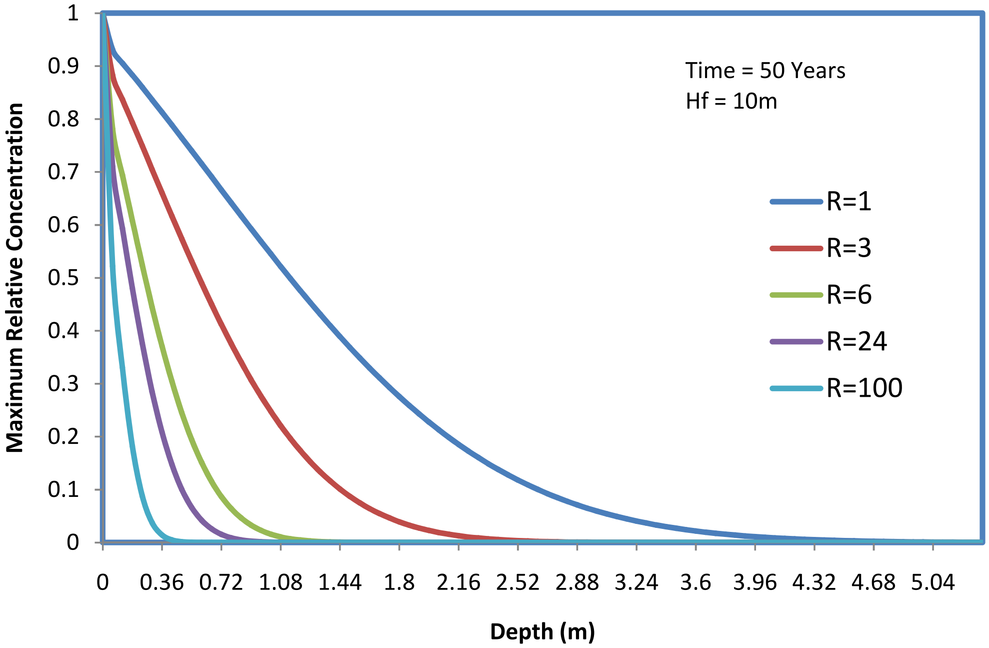


Fig. 5.38 Variation of Maximum Relative Concentration [R=1] [Nonconservative contaminant]Permeability=0.3cm/yr

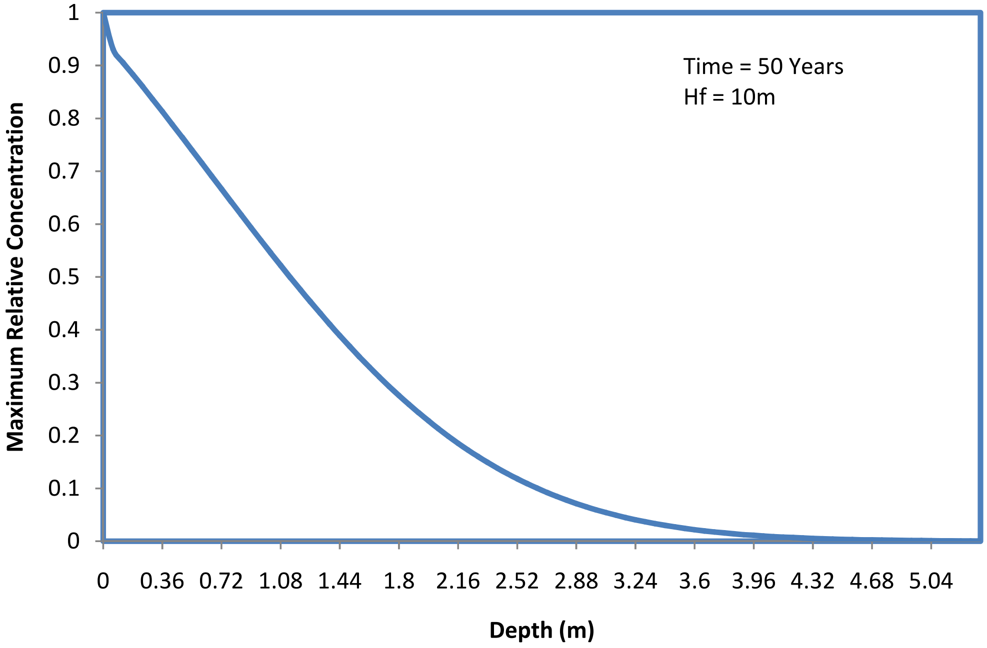


Fig. 5.39 Variation of Maximum Relative Concentration [R=3] [Nonconservative contaminant]Permeability=0.3cm/yr

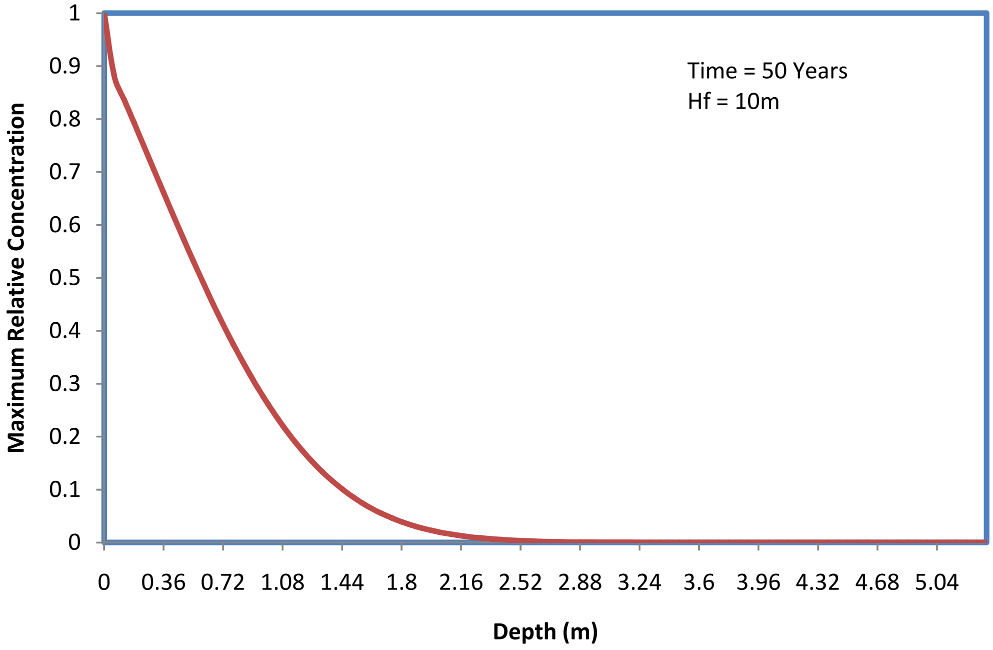


Fig. 6.40 Variation of Maximum Relative Concentration [R=6] [Nonconservative contaminant]Permeability=0.3cm/yr

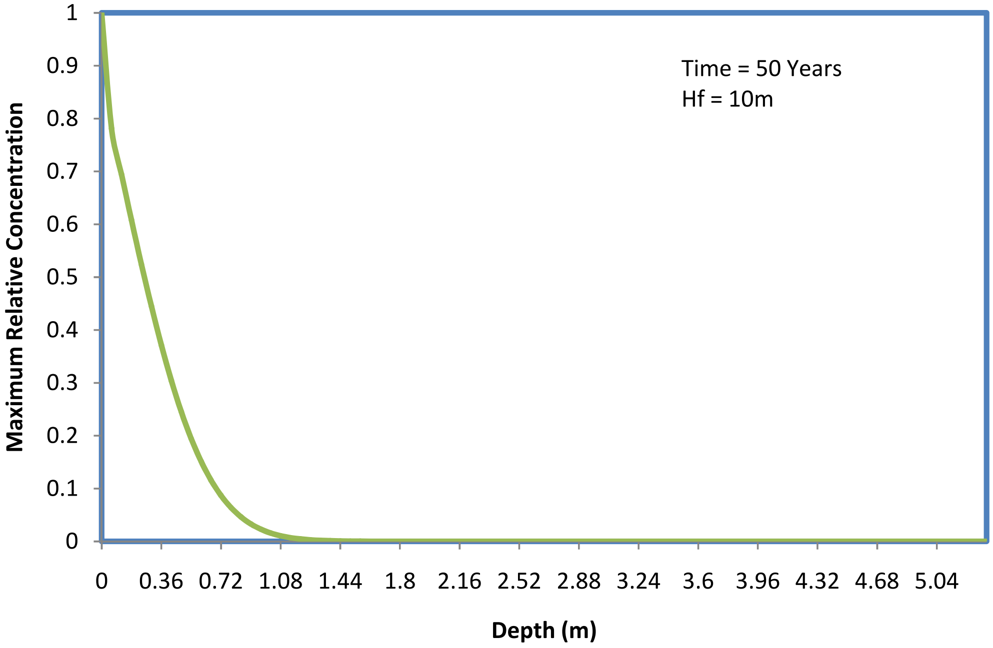


Fig. 5.41 Variation of Maximum Relative Concentration [R=24] [Nonconservative contaminant]Permeability=0.3cm/yr

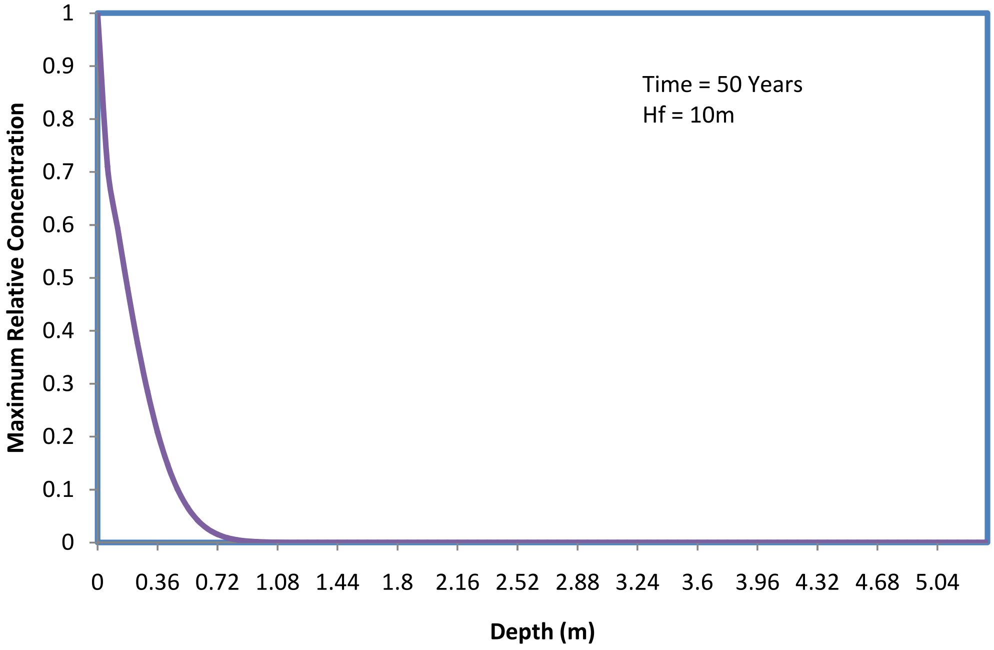


Fig. 5.42 Variation of Maximum Relative Concentration [R=100] [Nonconservative contaminant]Permeability=0.3cm/yr

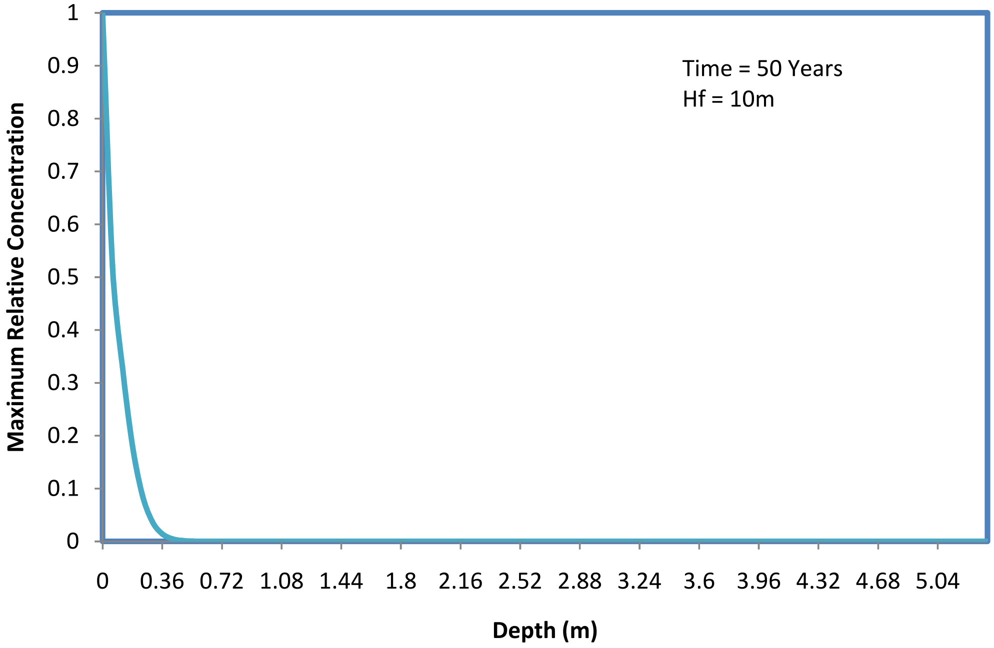


Fig. 5.43 Variation of Maximum Relative Concentration [Nonconservative contaminant]Permeability=3 cm/yr

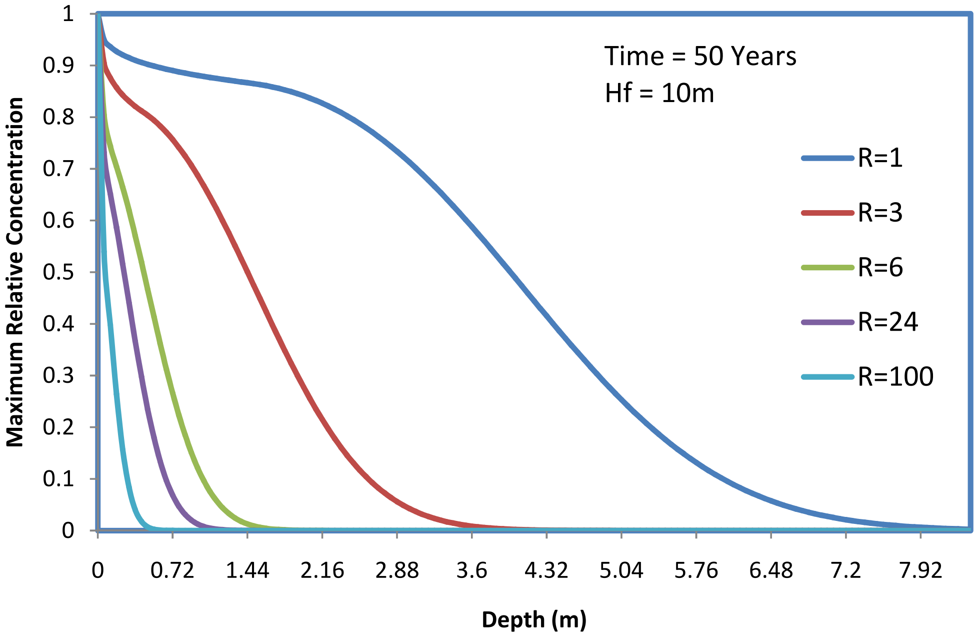


Fig. 5.44 Variation of Maximum Relative Concentration [R=1] [Nonconservative contaminant]Permeability=3cm/yr

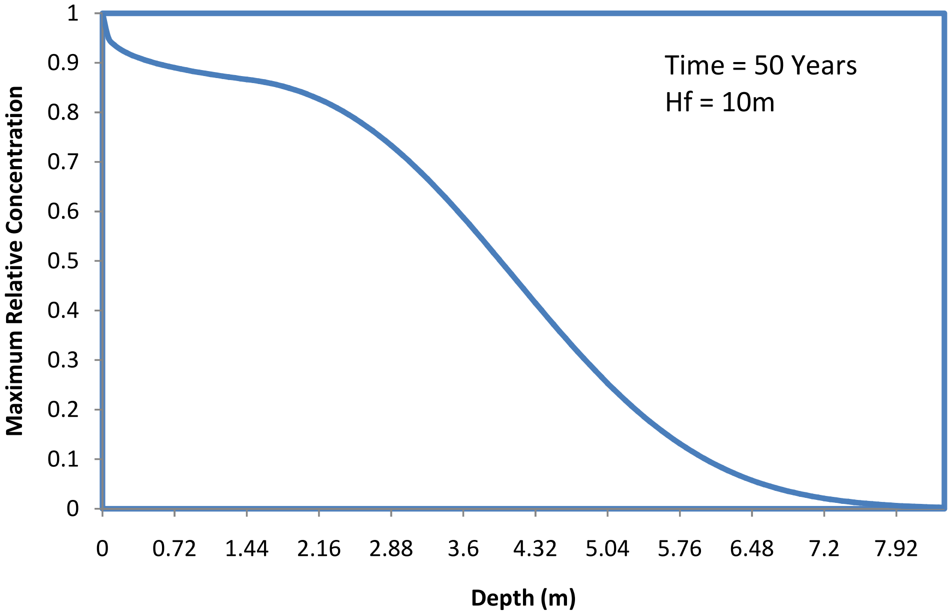


Fig. 5.45 Variation of Maximum Relative Concentration [R=3] [Nonconservative contaminant]Permeability=3cm/yr

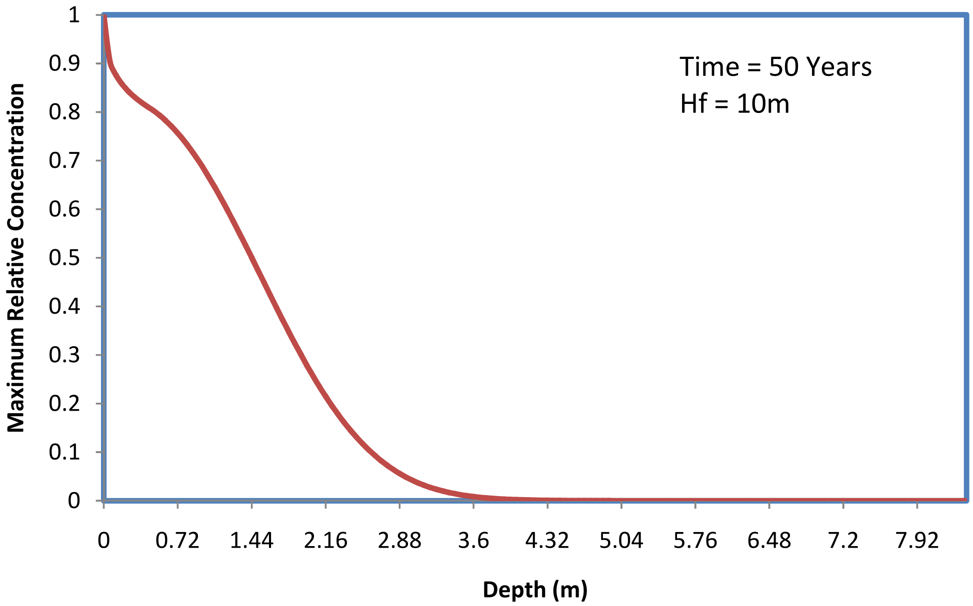


Fig. 5.46 Variation of Maximum Relative Concentration [R=6] [Nonconservative contaminant]Permeability=3cm/yr

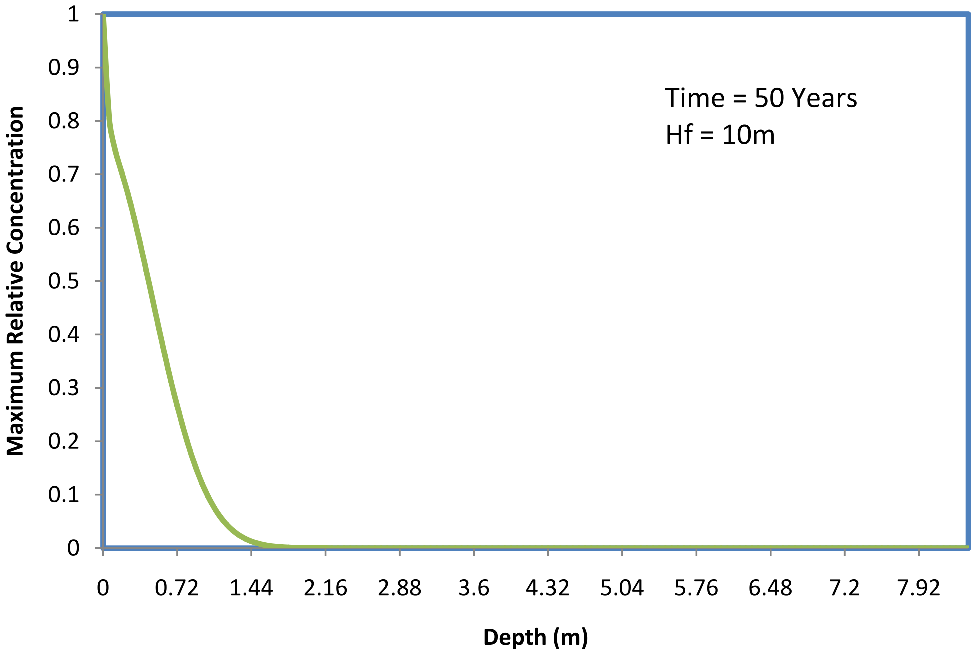


Fig. 5.47 Variation of Maximum Relative Concentration [R=24] [Nonconservative contaminant]Permeability=3cm/yr

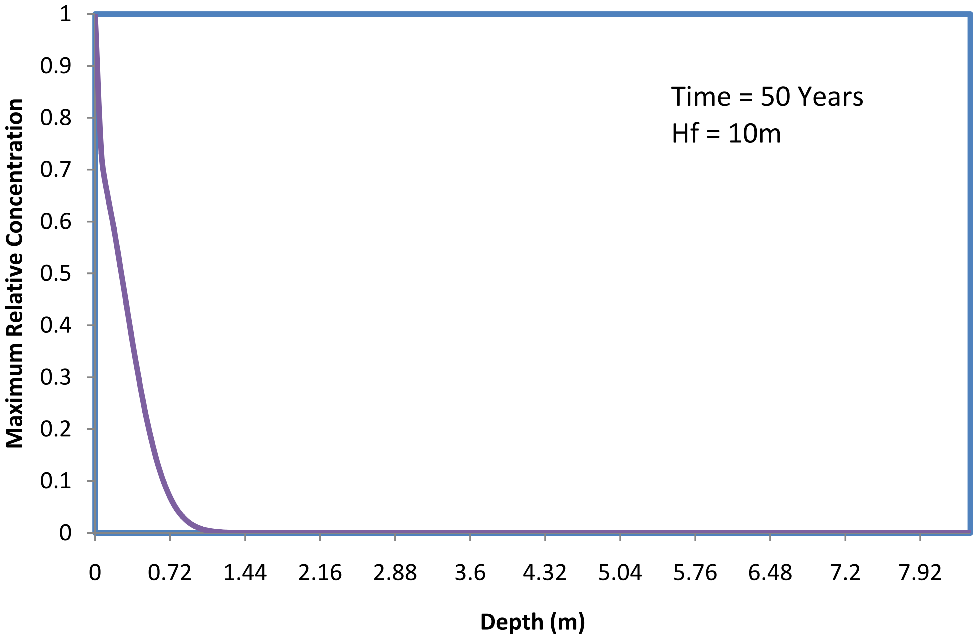


Fig. 5.48 Variation of Maximum Relative Concentration [R=100] [Nonconservative contaminant]Permeability=3cm/yr

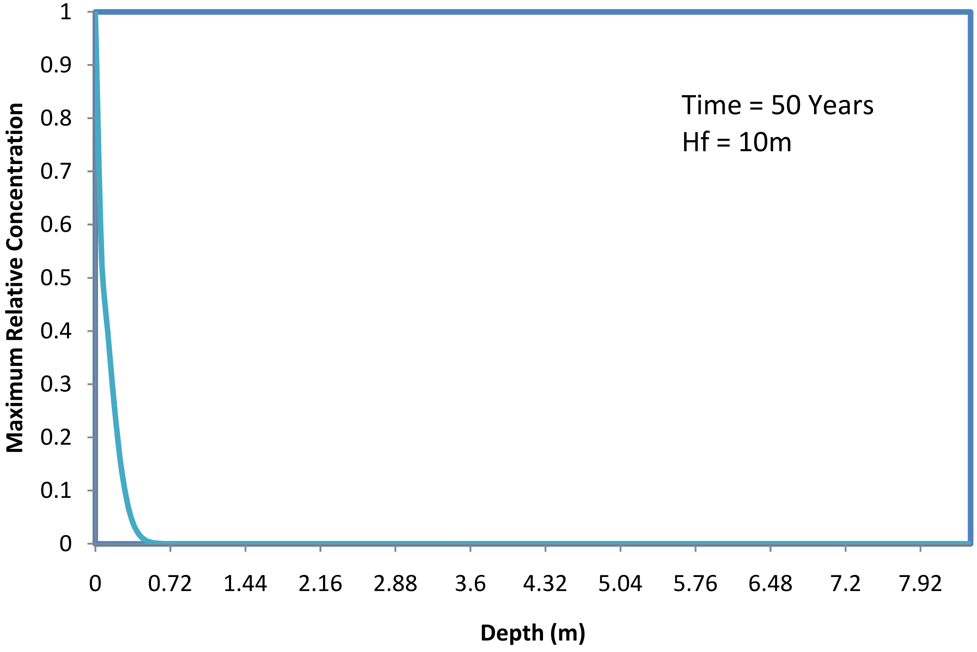


Fig. 5.49 Variation of Maximum Relative Concentration [Nonconservative contaminant]Permeability=6cm/yr

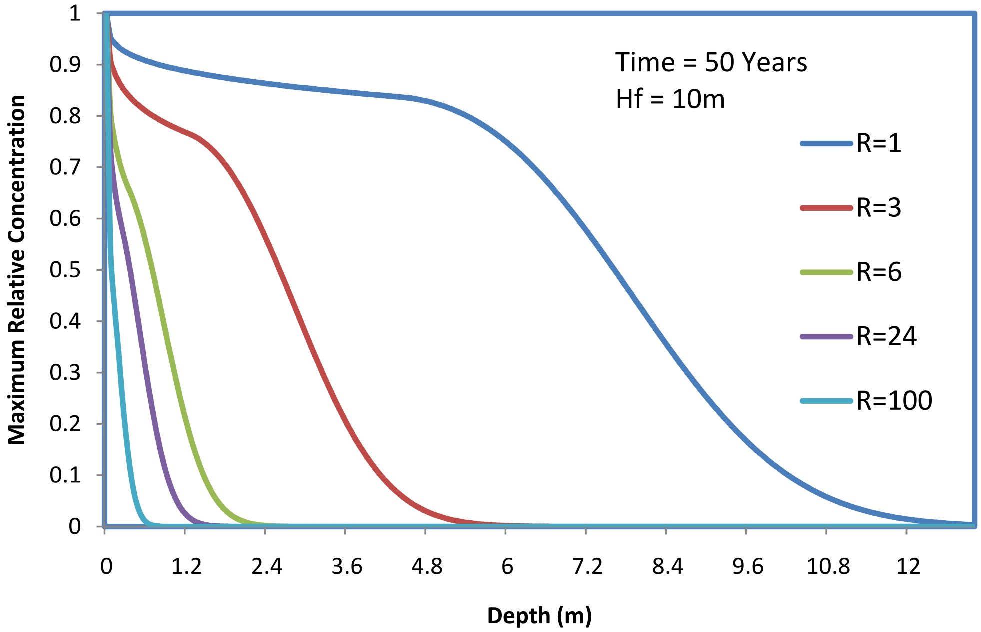


Fig. 5.50 Variation of Maximum Relative Concentration [R=1] [Nonconservative contaminant]Permeability=6cm/yr

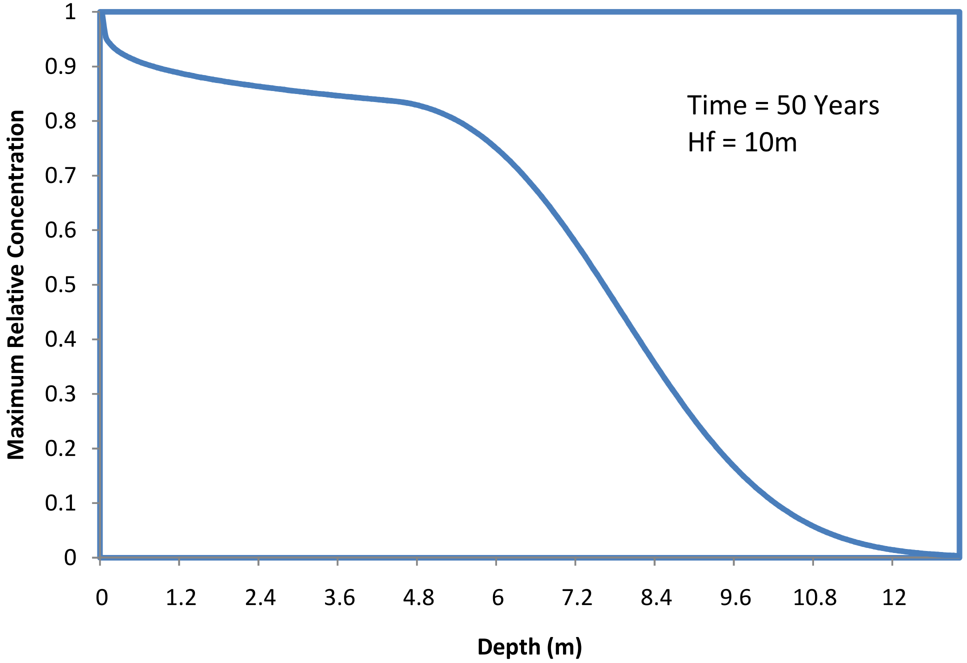


Fig. 5.51 Variation of Maximum Relative Concentration [R=3] [Nonconservative contaminant]Permeability=6cm/yr

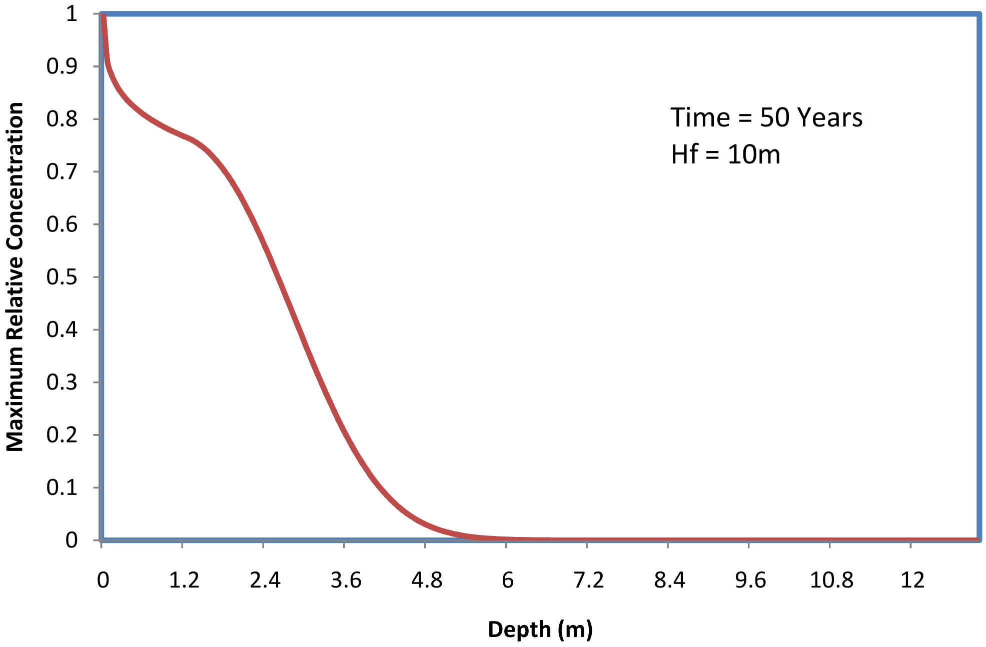


Fig. 5.52 Variation of Maximum Relative Concentration [R=6] [Nonconservative contaminant]Permeability=6 cm/yr

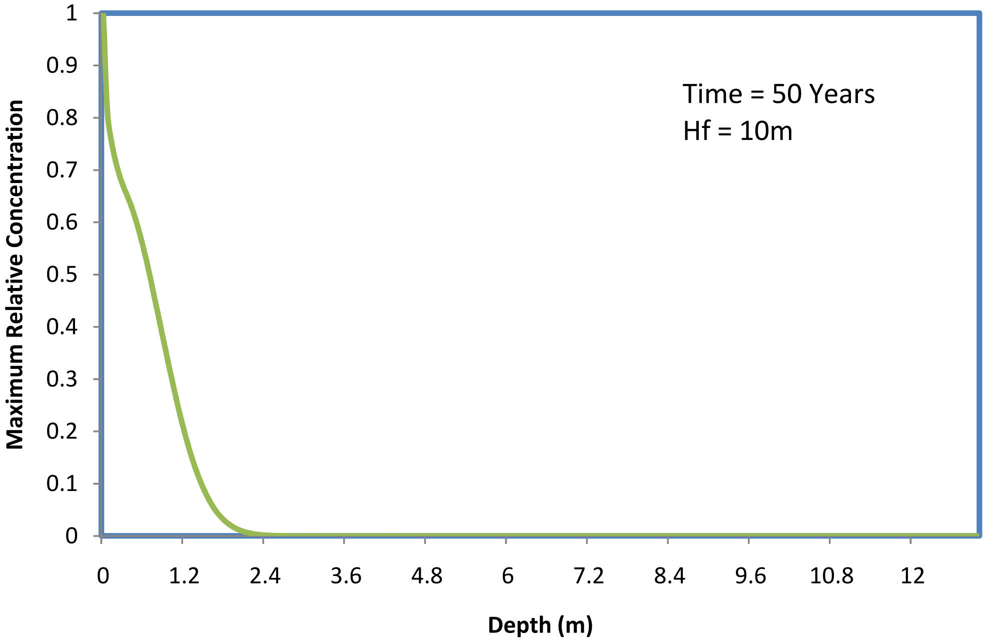


Fig. 5.53 Variation of Maximum Relative Concentration [R=24] [Nonconservative contaminant]Permeability=6cm/yr

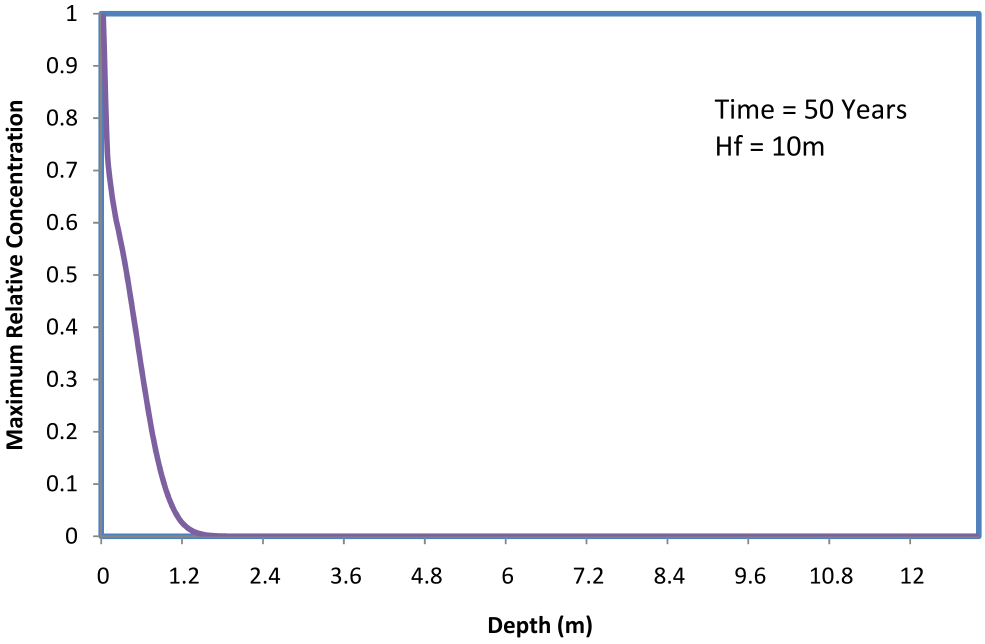


Fig. 5.54 Variation of Maximum Relative Concentration [R=100] [Nonconservative contaminant]Permeability=6cm/yr

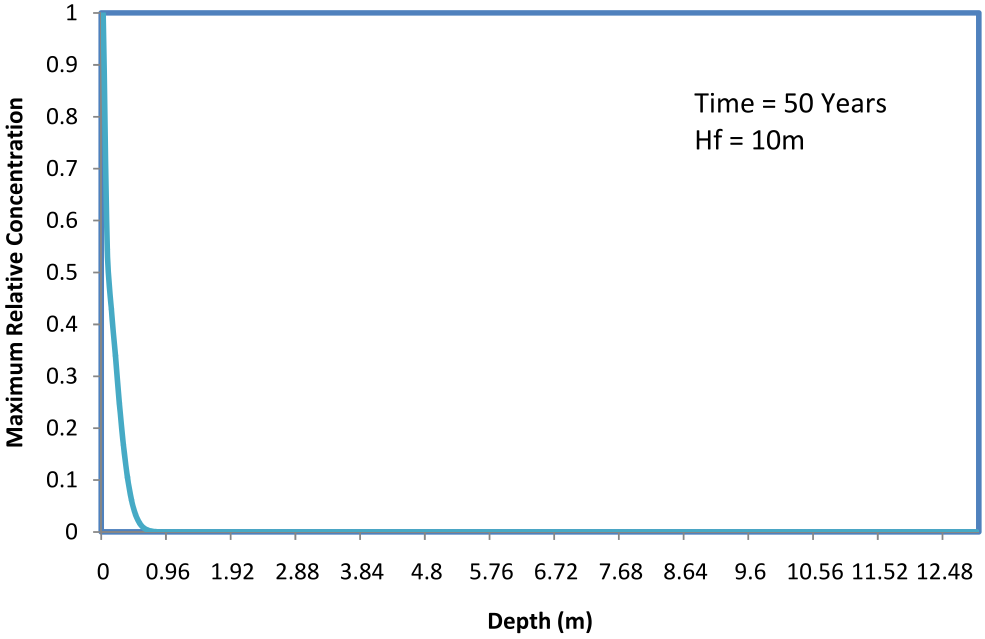


Fig. 5.55 Variation of Maximum Relative Concentration [Nonconservative contaminant]Permeability=10cm/yr

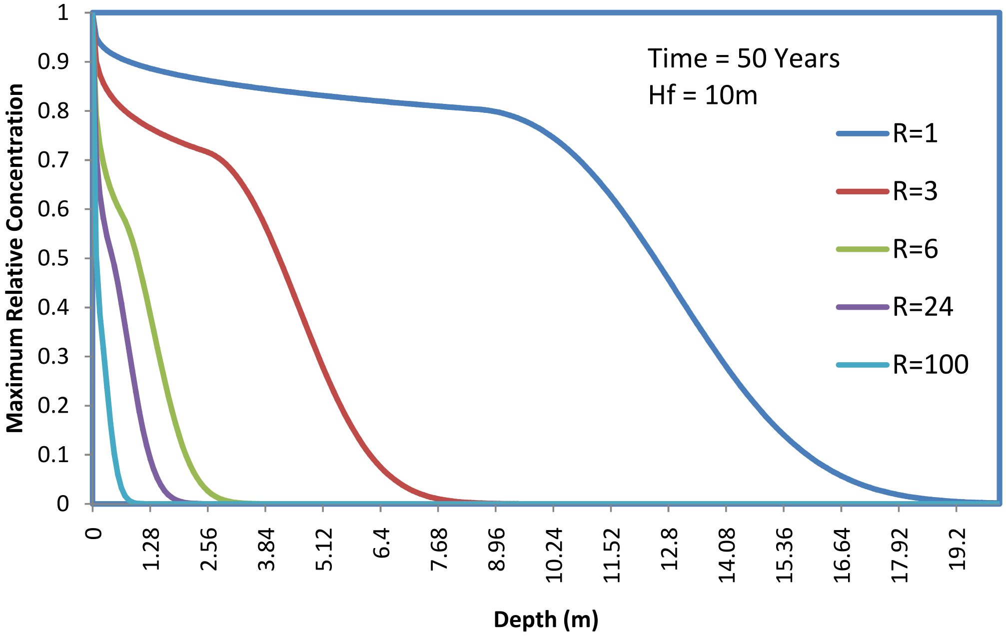


Fig. 5.56 Variation of Maximum Relative Concentration [R=1] [Nonconservative contaminant]Permeability=10cm/yr

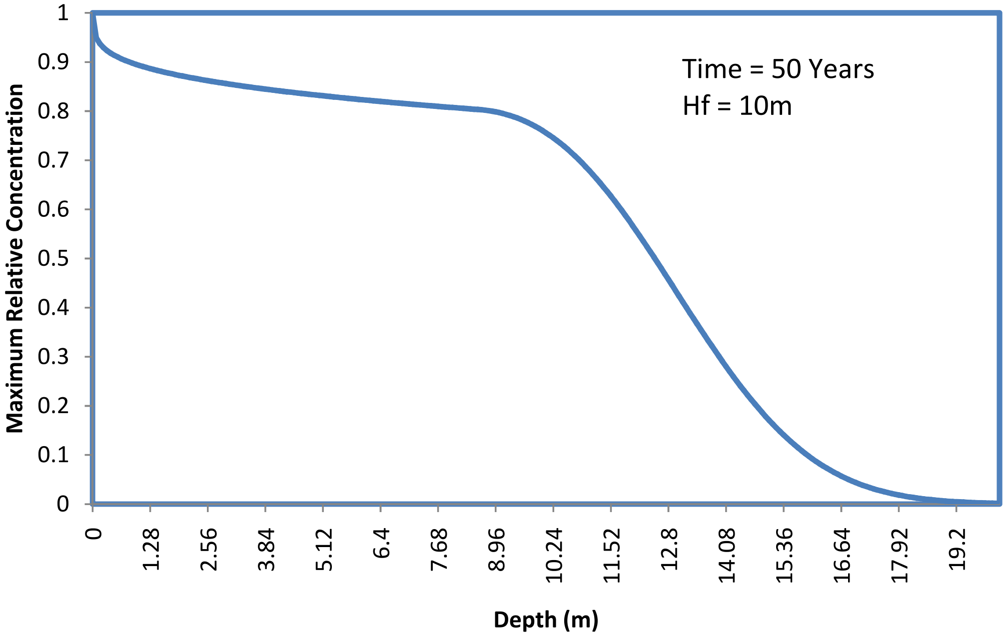


Fig. 5.57 Variation of Maximum Relative Concentration [R=3] [Nonconservative contaminant]Permeability=10cm/yr

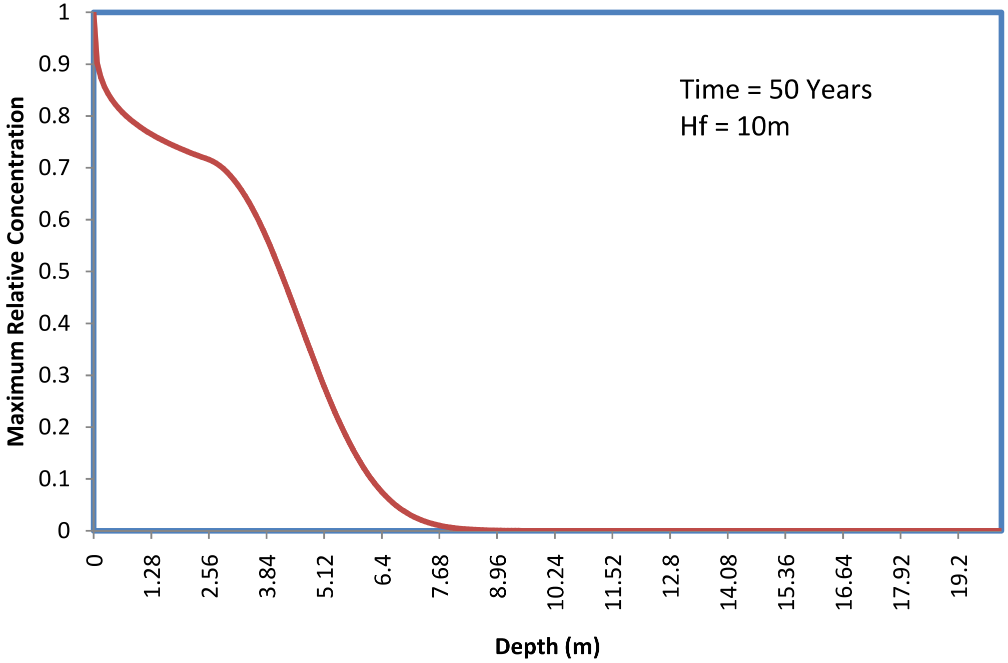


Fig. 5.58 Variation of Maximum Relative Concentration [R=6] [Nonconservative contaminant]Permeability=10cm/yr

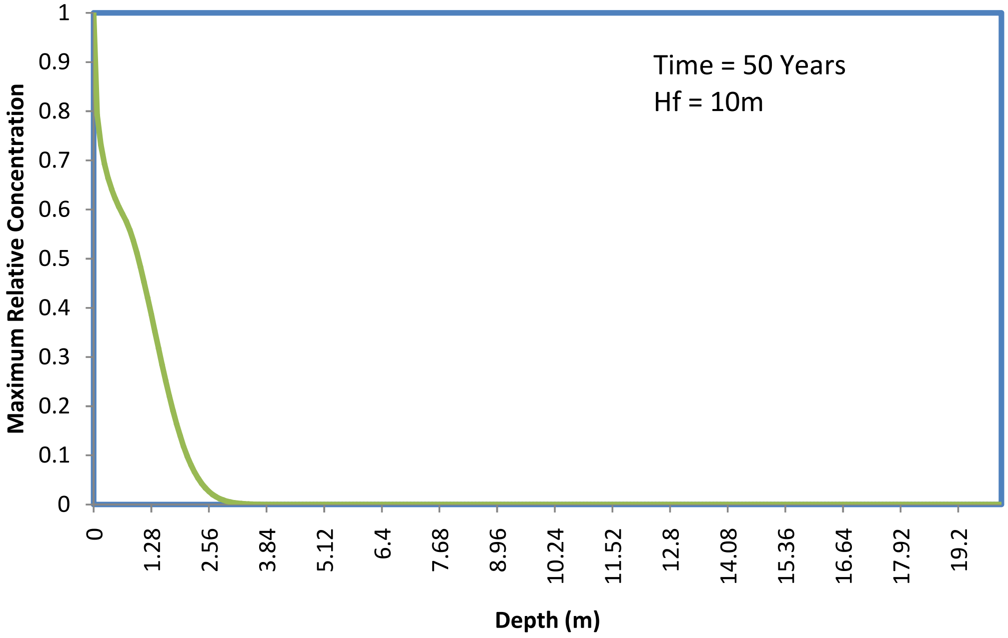


Fig. 5.59 Variation of Maximum Relative Concentration [R=24] [Nonconservative contaminant]Permeability=10cm/yr

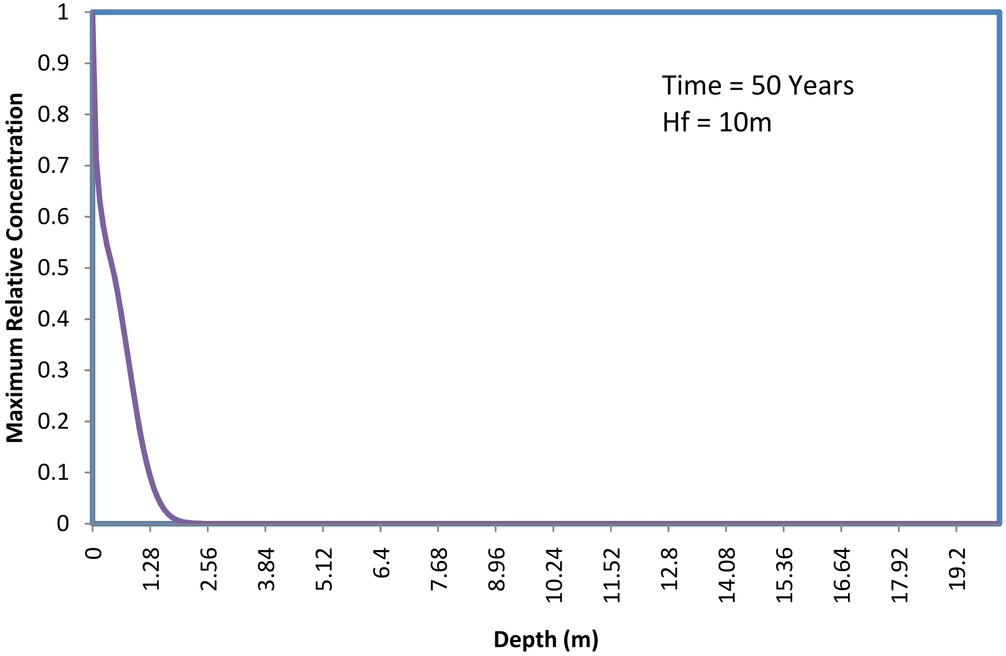
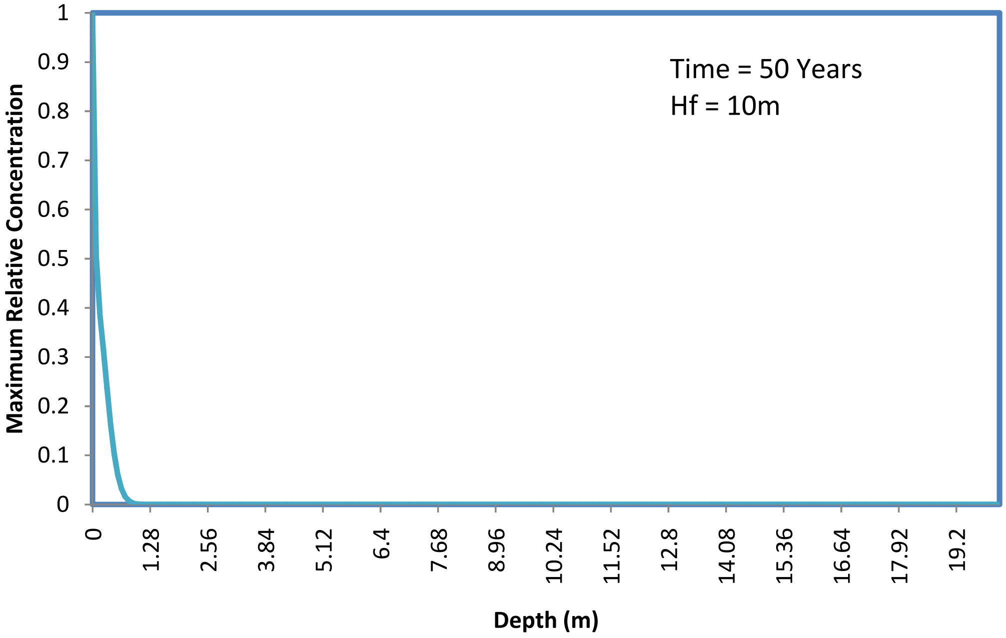


Fig. 5.60 Variation of Maximum Relative Concentration [R=100] [Nonconservative contaminant]Permeability=10cm/yr



Chapter 6

Evaluation of Various Admixtures for Landfill Liners

CHAPTER 6

EVALUATION OF VARIOUS ADMIXTURES FOR LANDFILL LINERS

6.1 Introduction

The materials for landfill liners commonly used are those which have low hydraulic conductivity. Ideally the materials to be used for the construction of landfill liner should be obtained from nearby locations or from the landfill site itself. But in many cases the locally available soil does not have low hydraulic conductivity. In such a case soil needs to be added with such materials which can impart it the desired properties, notably hydraulic conductivity. In certain cases the locally available soil may be added up with the materials to incorporate or improve the properties of landfill liner with regard to sorption and consequent retardation of the contaminant species of interest. A landfill liner materials composed of more than one materials may still act as homogeneous barrier, and in such case the combination of materials may be referred to as 'admixture'. Such a liner is supposed to have a larger contaminant retardation that that of natural soil as landfill liner.

6.2 Organoclays for landfill liners

Landfill leachate from municipal solid waste (MSW) and hazardous waste disposal facilities contain a wide range of potential environmental contaminants. In order to protect underlying groundwater resources from these contaminants, waste disposal sites are generally lined with compacted clay or a composite liner consisting of compacted clay and geomembrane. The design of these liner systems has focused on minimizing the flow rate of leachate through the liner. Generally compacted soil liner in the thickness of

1 to 1.2m is considered adequate to minimize the rate of contaminant migration by reducing the advective transport of contaminants through the liner. However, even with hydraulic conductivities less than 10^{-7} cm/s, the migration of many organic contaminants through compacted clay or composite liners by simple Fickian diffusion can be significant (Shakelford 1991; Park and Nibras 1993; Gullick 1998). Diffusion can result in early breakthroughs (less than 30 years) and large contaminant fluxes in compacted clay liners. Diffusion involves movement of a chemical species from a region of high concentration to a region of low concentration. With a liner system put into place, the only way to reduce the effect of diffusion is to reduce the concentration gradient by contaminant retardation. To overcome the inability of landfill liners to impede diffusive transport, enhancement of the sorption capacity of earthen liner materials becomes essential to improve liner performance. Enhancement of sorption capacity of landfill liner can be obtained by adding organoclays to the liner materials. The magnitude of organic contaminant sorption to natural soils and clays is related to the solubility of organic contaminants and the organic carbon content of the sorbent (Chiou 1998; Bartelt Hunt et al. 2003).

Sorption efficiency of amended clays for organic contaminants increases with the increase in number of carbon atoms associated with Quaternary Ammonium Cations (Jaruwong and Wibulswas 2003). The natural soils and clays typically have organic carbon contents less than 0.5%. Enhancement of organic contaminant sorption and thus minimizing the migration of leachate contaminants through earthen liners can be achieved by amending the earthen liners with materials capable of strongly sorbing organic contaminants.

The replacement of inorganic cations such as sodium in clay minerals by quaternary ammonium cations through the cation exchange reaction reduces the hydration of the clay and decreases its surface negative charge. Such replacement results in change in properties of the clay change significantly from being highly hydrophilic to increasingly organophilic. The sorption characteristics of clays amended by quaternary ammonium compounds, called organoclays to ionic and non-ionic organic compounds are mainly due to the partitioning process or hydrophobic interactions (Lo et al. 1998; Budhu et al. 1997). Organoclays effectively minimized the impact of phenolic wastes on cement hydration (Lo and Liljestrand 1996). The study on organoclays as waste containment barriers for pollutant attenuation have demonstrated it to be a attractive alternative to the conventional clay liners [Xu et al. 1997; Lo et al. 1997]. Increasing the sorption capacity of the earthen liner effectively reduces the contaminant advection velocity and the rate of solute diffusion during transient solute transport without requiring an increase in liner thickness (Lo 2003; Gullick and Weber 2001). Although a sorbed contaminant will eventually desorb from the liner, the mass flux from the bottom of the liner is significantly reduced when an organobentonite is a component of the liner (Bartelt Hunt et al. 2003).

6.3 Materials and Methods

Various admixtures were prepared and subjected to laboratory investigations with the objective of investigation the potential use of organoclays with soil-bentonite admixtures in retarding the movement of contaminants while maintaining required permeability specifications. Such amendment is likely to have impact on the minimum thickness required. In order to evaluate the potential effectiveness of sorptive materials as amendments to clay liners, sorption and permeability tests were performed on admixtures obtained by adding varying amount of the organoclay to the natural soil, and purified bentonite. One-dimensional transport modeling was performed to determine how effective each of these admixtures would be at retarding the transport of organic contaminants if used as landfill liner. The required liner thickness is also determined using design charts developed from a one-dimensional contaminant transport model.

6.3.1 Liner Materials

For the purpose of study, organoclay was obtained as synthesized organoclay using cationic surfactant; and organoclay as a manufactured product. Synthesized organoclay used in the study was obtained by modifying the purified bentonite by addition of coco dimethyl benzyl ammonium chloride. Small amount of ultra pure water was first added to the bentonite clay. The clay was then added with the coco dimethyl benzyl ammonium chloride with more amount of ultra pure water. Entire contents were mixed in a pug mill in laboratory. Mixed contents were kept in oven and a temperature of 102 ^{o}C was maintained for 48 hours followed by grinding and sieving of the contents. Organoclay as a manufactured product was obtained from M/s Cutch Oil & Allied Industries, Mumbai. The natural soil selected to be used in this work was locally available soil in Delhi, having yellowish color. The natural soil as obtained was oven dried at 103°C for three days, and then passed through a 300 micron sieve after breaking the clods. The soil passing through the sieve was collected and used in the experimental work. Purified bentonite used in this study was saturated with sodium. Both synthesized organoclay and manufactured organoclay product along with purified bentonite and natural soil were mixed in varying proportions to obtain the admixtures suitable for use as landfill liner material for the purpose of conduct of laboratory studies and transport modeling investigations.

Bentonite is a clay mineral with expansive characteristics and low permeability where montmorillonite is the main mineral. Montmorillonite swells when contacted with water approximately 900% by volume or 700% by weight. When hydrated under confinement, the bentonite swells to form a low permeability clay layer with the equivalent hydraulic protection of several feet of compacted clay when used in traditional landfill applications. Bentonite with soil has traditionally been used for landfill liner. High susceptibility of soil-bentonite mixtures to changes in hydraulic conductivity on exposure to chemical solutions, and low organic sorption capacity of bentonite, limit their use as barriers for waste containment. Amendment of bentonite clays with quaternary ammonium compounds results in exchange of cations and thus increase in basal spacing of such clays. The quaternary ammonium compounds function as pillars within the interlamellar region or intercalated layers of clay platelets, resulting in significant increase in the available adsorbent pore space.

In this study, eight soil-bentonite-organoclay admixtures (M1, M2, M3, M4, M5, MC_1, MC_2, MC_3) were prepared using the synthesized organoclay and manufactured organoclay. Organoclay, natural soil, and bentonite were mixed in different proportions. Composition of the admixtures is shown in Table 6.1. Percentage of each constituent was taken on a dry weight basis.

Table 6.1 Composition of Admixtures

Admixture	Percent Component (%)		
	Bentonite	Organoclay	Natural Soil
M1	0	25	75
M2	10	15	75
M3	15	10	75
M4	20	15	65
M5	20	20	60
MC_1	0	20	80
MC_2	5	15	80
MC_3	10	10	80

6.3.2 Compaction Properties

Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) for the five types of admixtures were determined with Proctor Test Method in accordance with BIS: 2720 (Part VII). The tests specimens prepared using admixtures were allowed to hydrate for 24 hours. Initial weight was recorded with empty proctor mould. The specimens were compacted in three layers in a 100 mm diameter mold and 127 mm height. Each layer was given 25 blows of 2.6 Kg weight rammer dropped from a height of 310 mm. Final weight was recorded with compacted specimen and Increase in weight was used to compute the increase in density achieved. The process was repeated with increased water content. At the end of each step, a small quantity of specimen was taken for the determination of moisture content. Moisture content of specimen at each step was determined using infra red moisture analyzer. For the determination of bulk dry density and porosity, the soil specimen was oven-dried for 48 h at 105°C. The mass and volume of the soil specimen were measured for calculating the bulk dry density and porosity.

6.3.3 Permeability of the admixture for landfill liners

Both consolidation tests and flexible wall permeability methods were used for the determination of permeability of the admixtures for landfill liners. The detail methods are discussed below:

6.3.3.1 Consolidation test

Hydraulic conductivity was determined by conducting consolidation test on the specimen to yield the values of coefficient of volume change m_v, and coefficient of consolidation c_v. The tests were carried out on the sample of 60mm diameter and 20mm thickness using standard consolidometers according to IS 2720 (Part XV). The initial water content of the samples was adjusted to the liquid limit. The inside of the ring was smeared with a very thin layer of silicon grease in order to avoid friction between the ring and soil sample. Filter paper was placed at the bottom and top of the sample. A top cap with a porous stone was placed above the soil sample. The entire assembly was placed in the consolidation cell and positioned in the loading frame. The consolidation cells were allowed to equilibrate for 24 hours prior to commencing the test. All the samples were initially loaded with a initial stress of 30 kPa, increasing by a ratio of 1 to a maximum pressure of 240 kPa. From the consolidation test result, a time-settlement curve was obtained at each pressure increment. The coefficient of consolidation c_v was obtained using Taylor's square root time ($\sqrt{T_v}$) method. The coefficient of permeability, k, was calculated by the equation $k=c_v m_v \gamma_w$ for various pressure increments using the c_v, and coefficient of volume m_v, where, γ_w is the unit weight of the pore fluid.

6.3.3.2 Flexible wall permeability

Hydraulic conductivity of the admixtures specimens were determined using Flexible wall permeameter. The tests were conducted on compacted admixture samples. The equipment used for the purpose was of make Humboldt. The diameter of the sample was 100mm.

Individual samples of the admixtures were thoroughly mixed at optimum water content and left for 24 hours period for their complete hydration. The sample were then compacted using Proctor test method as per IS:2720 (Part VII). Compacted specimens were then extruded, so that the remaining portion in the mold is 60 mm thick. The extruded portion of the compacted specimen was cut and removed using a hacksaw. The schematic of flexible wall permeameter used in this study is shown in Fig. 6.1. The specimen is placed between top and bottom caps in a cell filled with water. A latex membrane is used to seal the specimen between the caps and to isolate the specimen cell water. Vacuum suction is used to slide the latex membrane around the specimen. Tubing is connected to top and bottom caps for flowing water through the specimen. Water in the cell is used to apply confining pressure to the specimen and also to ensure that tight contact of membrane with the specimen, so as to prevent flow along the interface between the specimen and the membrane. All the fittings were tightened in such a way that they produce a complete leak-proof permeability cell and prevent water penetration into the test specimen. The weight of the specimen was measured to calculate the total volume available for the permeant to flow through.

6.3.2.2.1 Sample saturation

Back pressure saturation was used as outlined in ASTM D5084 to ensure full saturation. The permeability cell was also filled with tap water through which the confining pressure was applied to the specimen. After opening the vent valves connected with the cells and permeability cell, the valves that control flow through the specimen were opened so that the water can flow into the specimen and the air bubbles can be flushed out of the specimen. All the vent valves were then closed and air pressure was applied to the cells as well as to the permeameter chamber gradually so as to maintain a pressure difference of 20 to 25 kPa (3 to 5 psi) between the confining pressure and cell pressures.

It is imperative that the pressure head in both influent and effluent cells are the same during the period of saturation, so that no flow occurs from one cell to the other. To achieve this, the same air pressure regulator was used to apply pressure to both of the cells through a bridge connection in the pressure panel. The pressure in the cells was raised gradually up to 415 kPa and was kept 20 to 35 kPa below that of the permeameter chamber during the whole period of saturation. Backpressure, applied to the specimen through the top and bottom platens, was not allowed to surpass the confining chamber pressure. Otherwise, the specimen could be damaged by swelling due to higher internal pressure. Air bubbles left in the specimen during compaction, and any gas which might have been produced due to the chemical reaction, were dissolved in water due to the high backpressure. This process of saturation by backpressure was continued for at least 24 hours to completely dissolve air or gaseous substance from the test specimen and to complete any chemical primary reaction that might occur with water and chemical compounds present on the surface of the ash materials. Maximum saturation of the specimen was considered to be accomplished when no significant drop of influent and effluent water levels was observed over a period of 2 to 3 hours.

6.3.2.2.2 Permeation phase

During the permeation phase hydraulic gradient is applied across the specimen and water is allowed to flow. Selection of the appropriate hydraulic gradient is of utmost importance in determining the suitability of flow rate so that no crack or channel is developed through the admixture sample specimen during the permeability experiment.

Several methods can be used to apply the hydraulic gradient; they differ in how the difference in total head across the specimen is controlled. The methods in D 5084 are (i) the constant head method (Method A), (ii) the falling-head method (Method B), and (iii) the falling-head rising-tail method (Method C). In this study, the falling head-rising head method (D 5084-Method. In this method, the total head on the influent end of the specimen decreases while the total head on the effluent end increases. Thus, the difference in total head across the specimen decreases during the test. The changes in the drop in total head are read directly off the burettes as changes in the water levels. The permeability (*k*) is computed using the following relation:

$$k = \frac{a_{in} a_{out} L}{At(a_{in} + a_{out})} \ln\left(\frac{h_1}{h_2}\right) \qquad \text{………………6.1}$$

In the present study, effluent cell pressure was reduced from its applied backpressure of 415 kPa (60 psi) during saturation to the required pressure level to develop the pre-calculated hydraulic gradients. A higher value of hydraulic gradient was chosen in order to complete the permeability test in shorter interval of time

The effluent cell pressure was reduced from its applied backpressure of 415 kPa (60 psi) during saturation to the required pressure level to develop the pre-calculated hydraulic gradients. Since the hydraulic gradient also depends on the difference in levels between influent and effluent in addition to the applied regulated pressure, the fluctuation of the actual hydraulic gradient has been accounted for in the calculations. The constant pressure difference of 20 kPa (3.0 psi) between influent and permeameter cell was maintained until the completion of the experiment in order to avoid piping and specimen collapse.

Permeation through the admixture specimen was continued until at least four consecutive values of hydraulic conductivity were obtained in which. Termination of the permeability test was carried out by gradually and simultaneously lowering the applied air pressure in the influent, effluent and permeameter cell to zero. Cell pressure was always kept higher than chamber pressure so that the confining rubber membrane does not separate from the specimen surface. Water was then forced out of the cell by slightly increasing the confining pressure through the base opening valve.

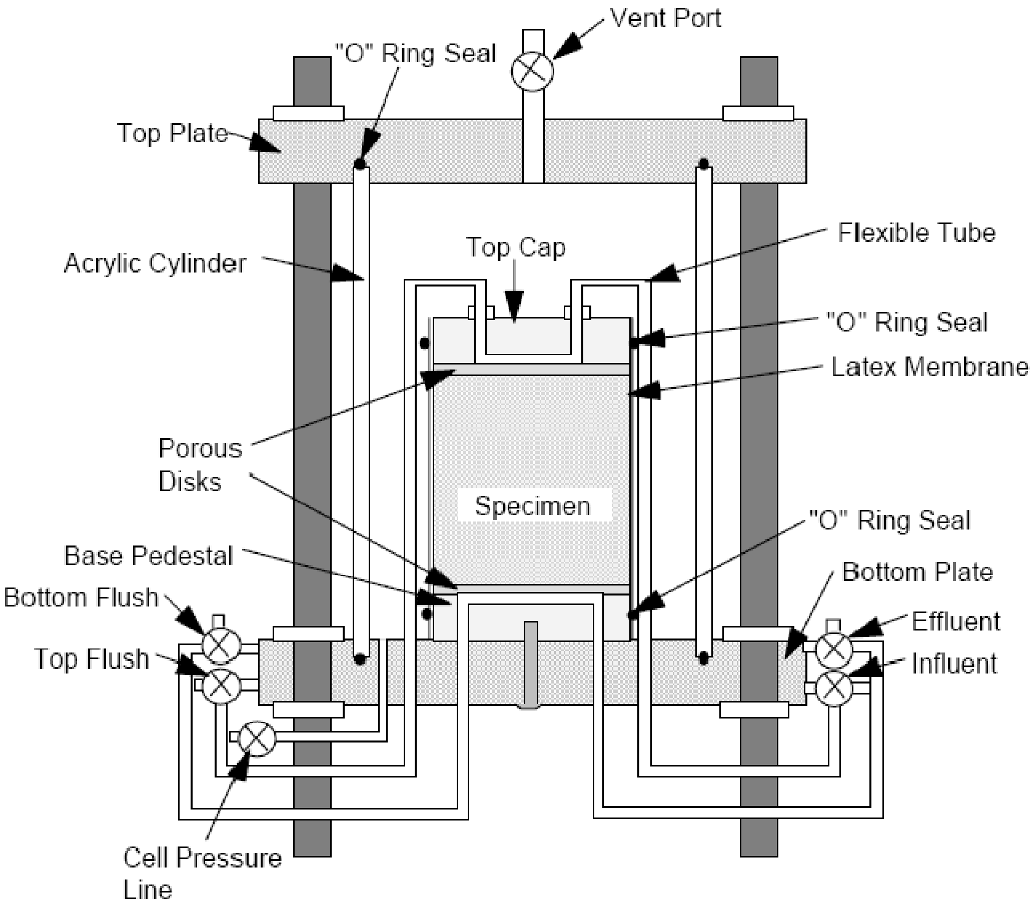


Fig. 6.1 Schematic Diagram of Flexible Wall Permeameter

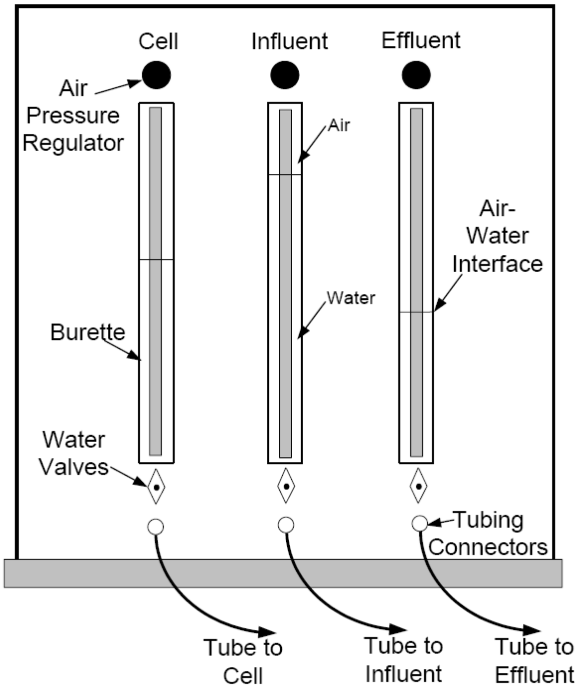


Fig. 6.2 Schematic Diagram of Pressure Panel

6.3.4 Sorption Tests

For assessing the organic sorption capacity of admixtures, the parameter Total Organic Carbon (TOC) was selected. The gross organic term TOC was selected to reflect the overall organic removal ability of admixtures. The admixtures were subjected to batch sorption tests for Total Organic Carbon (TOC). Percentage of each constituent was taken on a dry weight basis. TOC was determined using TOC analyzer of make analytikjena, model multi N/C 2100.

Organic sorption tests on admixtures prepared using synthesized organoclay were conducted using leachate obtained from Bhalaswa landfill site in Delhi. Similarly, organic sorption tests on admixtures prepared using manufactured organoclay were conducted using stock TOC solution of strength 5000 mg/L.

The TOC concentration in the landfill leachate was found to be 4980 mg/L. Batch sorption tests were carried out by diluting landfill leachate with ultra pure water to different levels varying from 10% to 80% strength. The initial concentration of TOC in these samples was thus varying from minimum of 498 mg/L to maximum of 3984 mg/L. Sorption studies were carried out on 1g, 2g, and 3g weight of each type admixture. Individual sample weights of admixtures were put into vial and 10 ml leachate diluted to various levels was added to each vial. In this way sorption studies were repeated on admixture weights of 1g, 2g, and 3g each. The vials were then mixed by subjecting them end-over-end rotation using a tube rotator for 24 hour. The supernatant (700 µL) was withdrawn, filtered, and placed into vials. Blanks contained 10 ml of 4980 mg/L of TOC solutions. The blanks in TOC showed equilibrium concentration of 4980 ± 2 mg/L, indicating that no significant loss of TOC in the blanks. For the stock solution of TOC, 10.627g of anhydrous potassium bipthalate ($C_8H_5KO_4$) was dissolved in carbon free ultra pure water and diluted to 500 ml (1.0 ml = 10 mg of organic carbon). Calibration curve was prepared by diluting the stock solutions to cover the range of TOC from 0 to 5000 mg/L using TOC analyzer. The experimental batch sorption data were fit to the nonlinear Freundlich isotherm $S = K_f C^b$, where S is the mass of solute removed from solution per unit mass of soil (mg/g), C is equilibrium concentration (mg/L), and K_f and b are the parameters determined from best fitted curve.

Similarly for the sorption studies on admixtures prepared using manufactured organoclays, were conducted using standard TOC solution. Standard solution for TOC was prepared by dissolving 10.627g of anhydrous potassium bipthalate ($C_8H_5KO_4$) was dissolved in carbon free ultra pure water and diluted to 500 ml (1.0 ml = 10 mg of organic carbon). The stock solution so prepared was diluted to various levels so as to obtain TOC concentration of 500, 1000, 1500, 2000, 2500, 3000, 3500, 4000 mg/L. Sorption characteristics were assessed by subjecting 1g, 2g, and 3g each of oven dried admixture to series of TOC solution prepared. For sorption assessment admixtures (1-3g) were placed in vials and TOC solution was added to it. The vials were then placed in tube rotator for 24 hours, after which supernant (500 µL) was withdrawn and tested for Equilibrium TOC concentration. TOC sorption was reported as difference of initial and final TOC concentration. Calibration curve was prepared by diluting the stock solutions to cover the range of TOC from 0 to 5000 mg/L. the experimental data obtained from such sorption tests is shown in Table 6.12 to 6.35. The experimental batch sorption data were fit to the nonlinear Freundlich isotherm $S = K_f C^b$ using software TableCurve 2D, where S is the mass of solute removed from solution per unit mass of soil (mg/g), C is equilibrium concentration (mg/L), and K_f and b are the parameters determined from best fitted curve.

6.3.5 Diffusion Tests

Diffusion cell used in this study was obtained by making certain modifications to rigid wall permeameter. The equipment allows high storage capacity of the source solution for continuing the diffusion test for a long time without affecting the quality and concentration of the solution. Both source and receptor containers are transparent thus allowing the fluid level to be monitored during diffusion and refilled if necessary to maintain a constant level. Diffusion setup as shown in Fig. 6.3 consists of inner aluminium mould and outer cylinder made of acrylic. The sample is placed in inside aluminum mould of diameter 75mm internal and compacted. Before placing the sample, bottom porous stone is kept. Top porous stone is placed once the sample has been placed and compacted. A threaded rod with piston at the bottom passes from the top plate of the setup. The rod is moved downwards and kept tight so as to avoid any bulging of the specimen. Any leakage from the top is prevented by placing a O ring within the groove and applying sufficient vacuum grease in and around the O-rings. The acrylic chamber allows viewing of the sample. The end plates were constructed of anodized aluminum. The receptor is a graduated pipette 1.25 cm internal diameter and 30 cm long. A cap is fitted at the open end of the pipette to prevent any ingress of impurities and evaporation from the solution. Receptor tube is graduated and the source chamber was marked so as to monitor the level of the fluid while the tests progresses. Diffusion of the contaminants from landfill leachate was carried out by keeping the liquid levels of both source chamber and receiving tube the same. A schematic of the diffusion cell is shown in the figure below which shows the relevant terms.

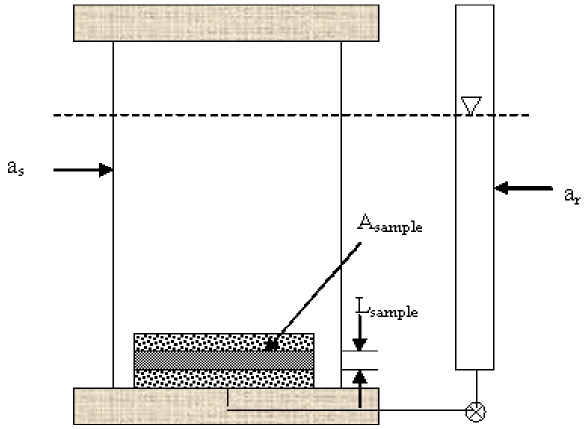


Fig. 6.3 Schematic Diagram of Diffusion Setup

A stainless steel ball valve connects at the bottom of the receptor tube and separates the solution in the receptor tube from source chamber and bottom porous stone. The valve is kept closed while collecting the sample for analysis from the receptor tube so that no disturbance or hydraulic gradient is created within the diffusion cell. Filter paper in between the specimen and porous stones was avoided to avoid any reaction between the constituents of the paper, chemical solutions, and clay minerals after a certain period of time. A highly concentrated solute flows from the source chamber towards the receptor tube with time due to the concentration gradient.

6.3.5.1 Sample preparation

The grooves of the base plate, fittings of the diffusion cell, and receptor tube are cleaned thoroughly. Sufficient amount of admixture is taken, and de-ionized water is added to it in sufficient quantity corresponding to its optimum moisture content. The sample is then allowed to hydrate for 24 hours before placing it in diffusion cell. After 24 hours, the admixture sample is placed in diffusion cell and top porous stone is placed over it. Outer Plexiglas cylinder is placed in its position with O rings at the top and bottom. The top plate is placed on the top end of the cylinder and tightened with screws. Landfill leachate collected from the landfill site is poured into the source chamber. The top plate is then placed and is tightened with the wing nuts so that no opening in the connection is available for air-flow. De-ionized water is poured into the receptor tube up to the same level as the source solution in the chamber. A cap is then placed on top of the receptor tube to prevent any air circulation or evaporation of the receptor solution during the process of diffusion.

6.3.5.2 Diffusion analysis

Diffusant samples were collected in chemically resistant bottles and mixed with 1% nitric acid for preservation at 4 ^{o}C in the refrigerator until the actual chemical analyses were done. The acidification is a required step in the preservation and chemical analysis of the samples, and does not interfere with the accuracy of the measurement in any way. The samples were analyzed for chlorides and TOC.

Transport of chemicals through a porous medium by dispersion consists of two processes, namely, molecular diffusion (commonly known as diffusion) and mechanical (or hydrodynamic) mixing. Diffusion is defined as the process whereby ionic or molecular constituents are transported under the influence of their kinetic activity in the direction of their concentration gradient.

Dissolved chemicals flow from the high concentration location to the low concentration location. The amount of mass flux, depends on its chemical concentration gradient. Chemical diffusion in soils is much slower than in the free solution because of the effect of porosity, especially in fine grained soils where the permeability is lower and where a tortuous pore channels exist. Transport of solute through porous medium is governed by Fick's law, which simply states that the rate of diffusion of chemicals is directly proportional to the concentration gradient. Fick's first law is only applicable for diffusive flux of solutes under steady-state condition when the concentration gradient within the medium does not change with time. During the process of solute transport due to diffusion and, the concentration of the solute is measured in the receptor chamber. Due to the process of diffusion under the action of concentration gradient of solute species of interest the concentration of the solute in the receptor chamber keeps on increasing. The concentration of solute in the receptor chamber is measured by collecting the sample and analyzing it at regular intervals. The diffusion coefficient from such analysis is computed using eth following expression

$$D_* = \frac{\Delta QL}{\Delta tnC_0} \qquad \text{………………6.2}$$

Where D*, is effective molecular diffusion coefficient for solute species of interest, ΔQ/Δt, is the increase in mass of the solute species of interest over the incremental time period Δt, *n* is the porosity of admixture sample, L is the length of sample, and C_0 is the concentration of solute species of interest in source chamber.

In the steady-state condition, the increase in concentrations for a given solute are converted to increase in mass of the solute in receptor chamber per unit cross-sectional area of the admixture sample. The cumulative mass of the solute species transported into the receptor chamber is thus given by the equation:

$$Q = \frac{1}{A}\sum_{i=1}^{n}\Delta m_i = \frac{1}{A}\sum_{i=1}^{n}\Delta C_i V_i \qquad \ldots\ldots\ldots\ldots\ldots\ldots 6.3$$

where, A = cross-sectional area of the specimen, Δm_i = mass increment of the solute species *i* collected over a time increment (Δt), ΔV_i = increment volume of the solution from which the outflow flux is collected, C_i = the concentration of the solute species in the incremental volume, and n = number of incremental samples (solution) collected during the total elapsed time, *t*. As the diffusion tests in this case have been carried out under zero hydraulic gradient the computed values of effective molecular diffusion coefficients may be referred to as hydrodynamic dispersion coefficients. The sample is de assembled after the completion of tests and subjected to oven drying for the further determination of porosity for the final determination of diffusion coefficients.

6.4 Results and Discussions

The plotted values of dry density against the moisture contents for all the admixtures studies are shown in in Fig. 6.1 to Fig. 6.8. The corresponding data of compaction tests are listed in Table 6.3 to Table 6.11. All the admixtures can be seen to be showing increase in dry density with increase in moisture content. The optimum moisture content was found to be ranging from a minimum of 21.9% to a maximum of 34.2 %.

The results of sorption plotted in the form of graph is shown in Fig. 6.12 to 6.35. All the sorption isotherms can be seen to be having convex shape. Such a curve is indicative of weak sorbent to sorbate contact at low concentration, but as the bonding between is achieved the sorption is followed in a co-operative manner. The sorption tests were conducted on 1, 2, and 3g sample of each of the admixture. It can be seen from the sorption isotherms in Fig. 6.11 to 6.34, that the results of sorption tests are scalable. Thus the results of sorption tests obtained from laboratory tests can be considered to be repeated in the field as well.

The hydraulic conductivity determined from consolidation tests data was found to be varying from a minimum of 7.4 x 10^{-9} cm/sec to a maximum of 5.4 x 10^{-8} cm/sec. The same determined from flexible wall permeability ranged from a minimum of 1.6 x 10^{-8} cm/sec to a maximum of 1.7 x 10^{-7}cm/sec. The results obtained from flexible wall permeability method, can be seen to be more conservative.

Table 6.2 Physical and Compaction Properties of Admixtures

Admixture	Bulk Dry Density (Kg/m^3)	Porosity	Optimum Moisture Content (%)	Maximum Dry Density (Kg/m^3)
M1	1242	0.44	24.2	1493
M2	1210	0.45	28.2	1362
M3	1158	0.46	30.6	1347
M4	1119	0.47	33.5	1288
M5	1104	0.47	34.2	1268
MC_1	1210	0.45	24.2	1493
MC_2	1202	0.45	21.4	1324
MC_3	1192	0.46	21.9	1335

Table 6.3 Compaction Data (Admixture M1)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1245
2	20	1378
3	25	1508
4	30	1312
5	40	1049

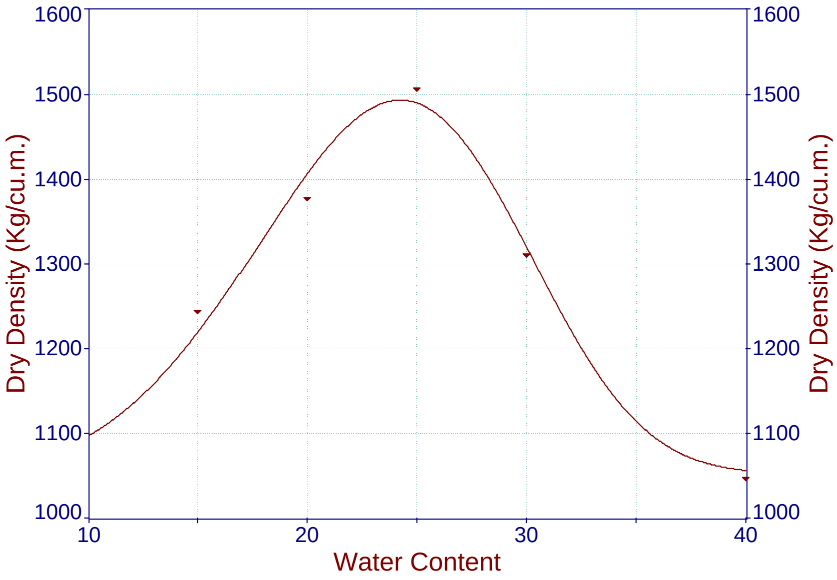


Fig. 6.4 Compaction Curve (Admixture M1)

Table 6.4 Compaction Data (Admixture M2)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1049
2	20	1248
3	25	1341
4	30	1362
5	40	1217

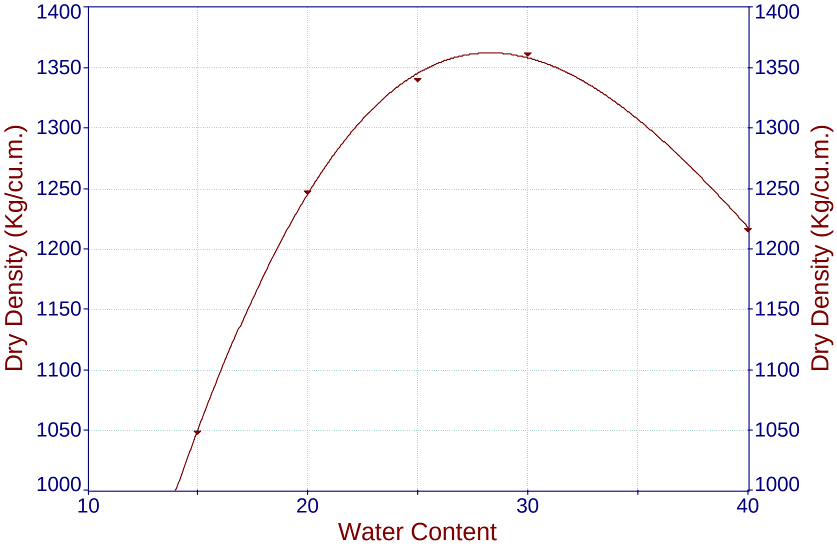


Fig 6.5 Compaction Curve (Admixture M2)

Table 6.5 Compaction Data (Admixture M3)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1054
2	20	1185
3	25	1314
4	30	1342
5	40	1267

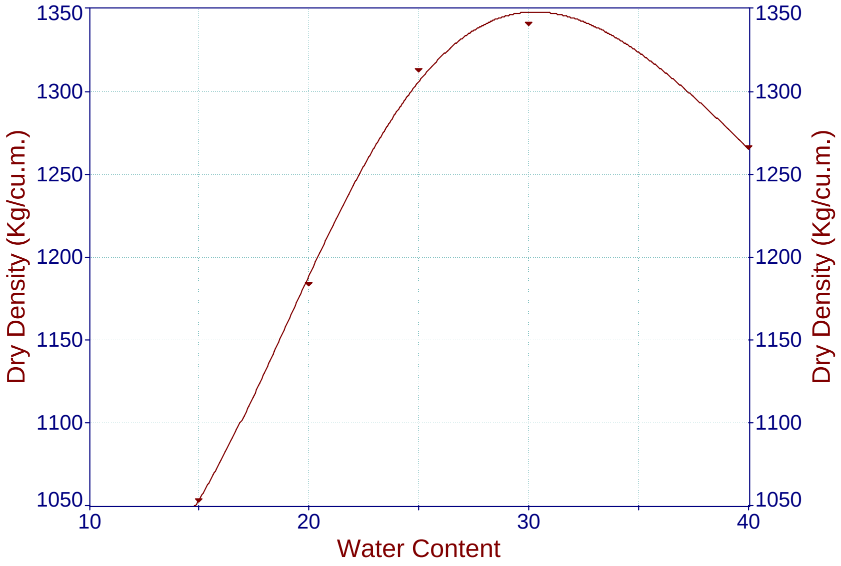


Fig 6.6 Compaction Curve (Admixture M3)

Table 6.6 Compaction Data (Admixture M4)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1029
2	20	1089
3	25	1167
4	30	1268
5	40	1240
6	45	1212
7	50	1158

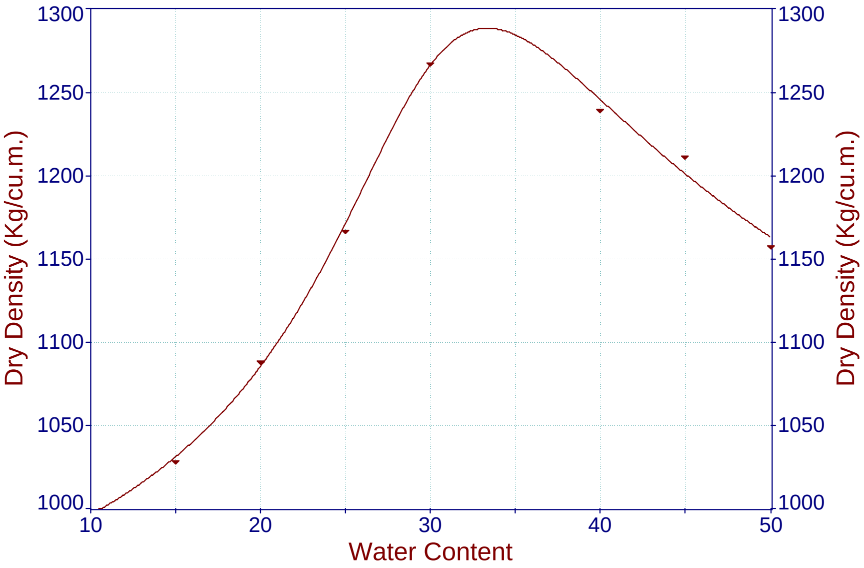


Fig. 6.7 Compaction Curve (Admixture M4)

Table 6.7 Compaction Data (Admixture M5)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1035
2	20	1095
3	25	1189
4	30	1242
5	40	1255
6	45	1225
7	50	1186

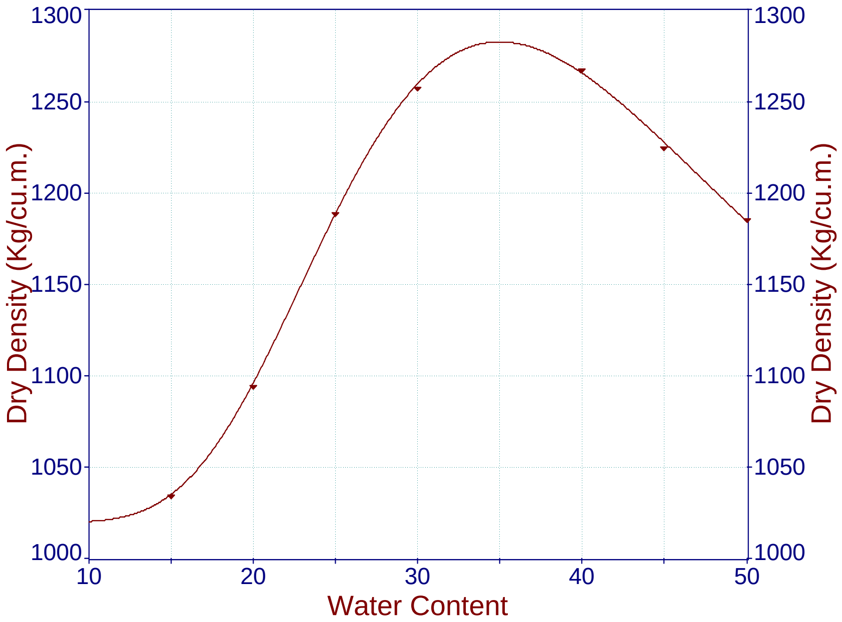


Fig. 6.8 Compaction Curve (Admixture M5)

Table 6.8 Compaction Data (Admixture MC_1)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1056
2	20	1298
3	25	1388
4	30	1345
5	40	1212

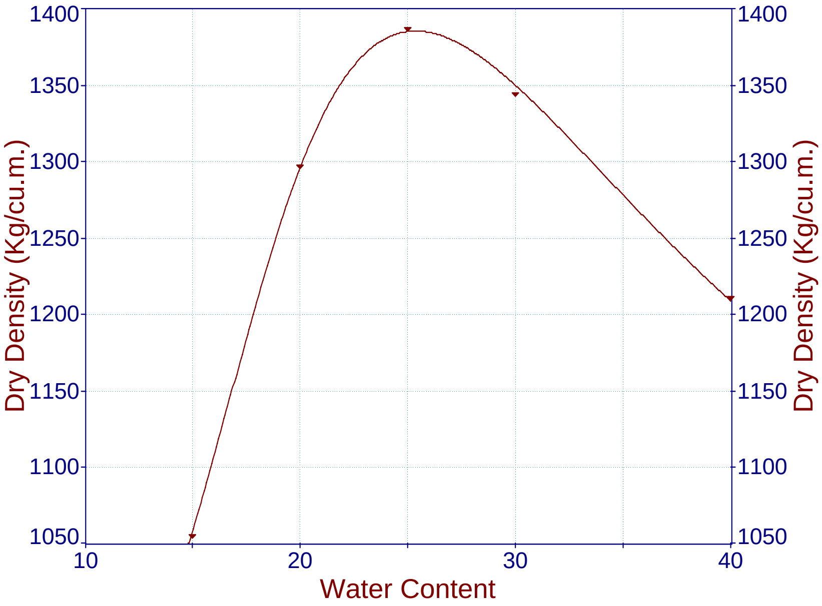


Fig. 6.9 Compaction Curve (Admixture MC_1)

Table 6.9 Compaction Data (Admixture MC_2)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1062
2	20	1311
3	25	1402
4	30	1289
5	40	1198

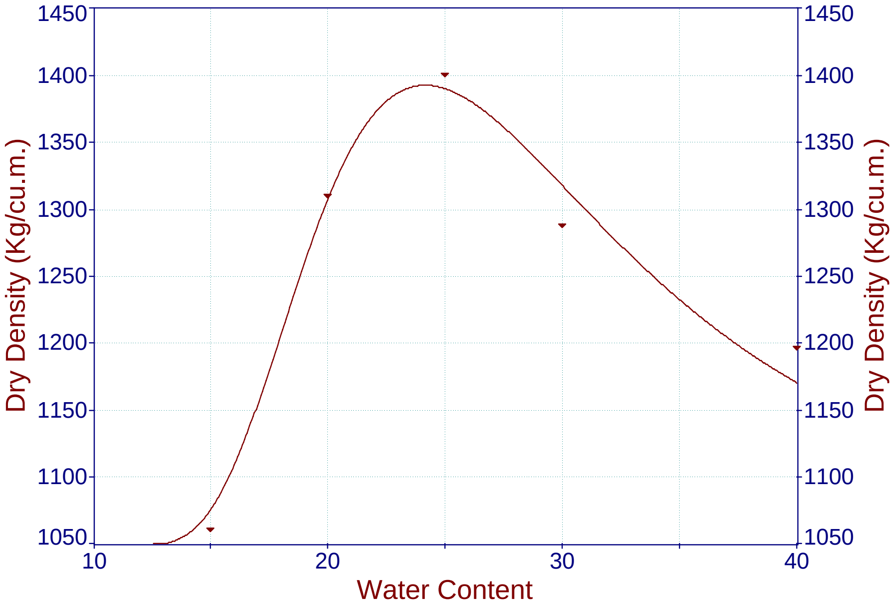


Fig. 6.10 Compaction Curve (Admixture MC_2)

Table 6.10 Compaction Data (Admixture MC_3)

S.No.	Water Content	Dry Density (Kg/m^3)
1	15	1078
2	20	1358
3	25	1405
4	30	1331
5	40	1202

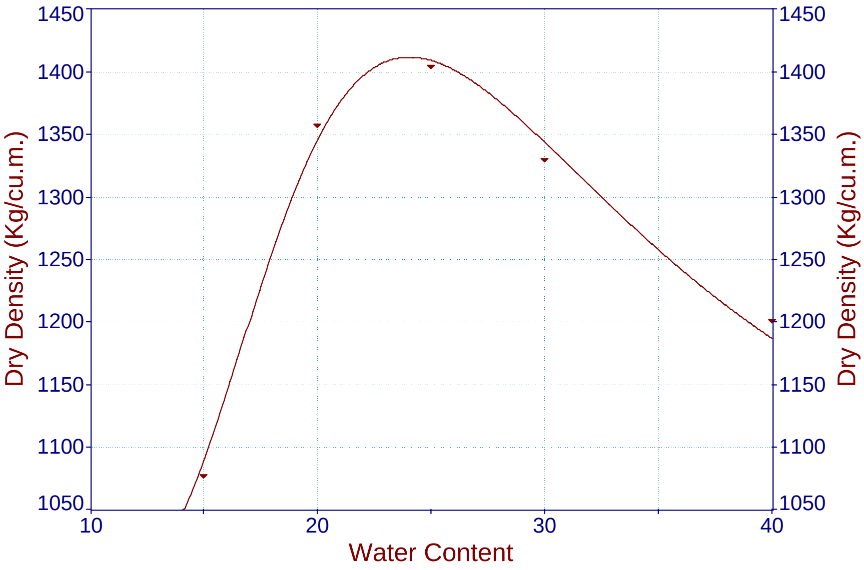


Fig. 6.11 Compaction Curve (Admixture MC_3)

Table 6.11 Consolidation Test Data

Admixture	Pressure (Ksc)	m_v	c_v	Hydraulic Conductivity (cm/sec)
M1	0.6	0.0156	0.0032	5.03E-08
M1	1.2	0.0145	0.0034	5.05E-08
M1	2.4	0.0135	0.0037	5.03E-08
M2	0.6	0.0116	0.0024	2.84E-08
M2	1.2	0.0108	0.0026	2.85E-08
M2	2.4	0.0093	0.0028	2.82E-08
M3	0.6	0.0076	0.0021	1.61E-08
M3	1.2	0.0075	0.0022	1.67E-08
M3	2.4	0.0070	0.0023	1.64E-08
M4	0.6	0.0050	0.0018	9.07E-09
M4	1.2	0.0046	0.0019	9.08E-09
M4	2.4	0.0045	0.0020	9.05E-09
M5	0.6	0.0043	0.0017	7.40E-09
M5	1.2	0.0041	0.0017	7.43E-09
M5	2.4	0.0040	0.0018	7.45E-09
MC_1	0.6	0.0093	0.0023	2.18E-08
MC_1	1.2	0.0086	0.0025	2.20E-08
MC_1	2.4	0.0079	0.0027	2.20E-08
MC_2	0.6	0.0073	0.0022	1.62E-08
MC_2	1.2	0.0068	0.0023	1.60E-08
MC_2	2.4	0.0065	0.0024	1.61E-08
MC_3	0.6	0.0070	0.0020	1.41E-08
MC_3	1.2	0.0066	0.0021	1.42E-08
MC_3	2.4	0.0061	0.0023	1.42E-08

Table 6.12 Hydraulic Conductivity of Admixtures (Flexible Wall Permeability)

Admixture	Permeability (cm/sec)
M1	1.4×10^{-7}
M2	8.0×10^{-8}
M3	4.7×10^{-8}
M4	1.8×10^{-8}
M5	1.6×10^{-8}
MC_1	1.7×10^{-7}
MC_2	9.1×10^{-8}
MC_3	8.7×10^{-8}

Table 6.13 Sorption Data for Admixture M1(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	993.9	2.1	2.1
3	1494	1488.6	5.4	5.4
4	1992	1984.5	7.5	7.5
5	2490	2473.2	16.8	16.8
6	2988	2956.3	31.7	31.7
7	3486	3442.5	43.5	43.5
8	3984	3922.1	61.9	61.9

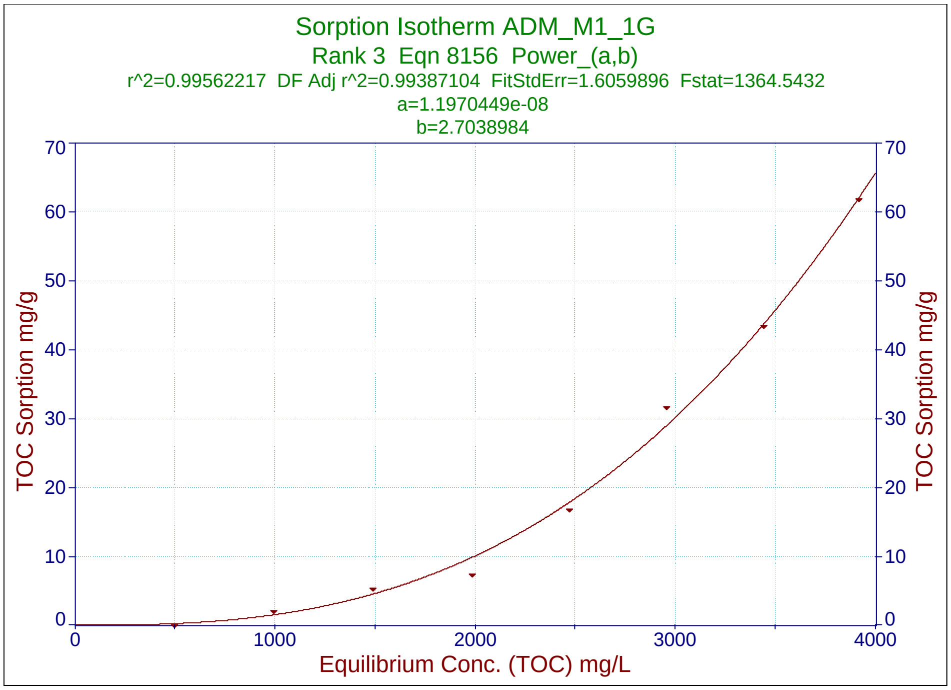


Fig. 6.12 Sorption Isotherm (Admixture M1_1g)

Table 6.14 Sorption Data for Admixture M1(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	992.1	3.9	1.9
3	1494	1482.8	11.2	5.6
4	1992	1976.6	15.4	7.7
5	2490	2457.7	32.3	16.1
6	2988	2925.9	62.1	31.0
7	3486	3400.7	85.3	42.6
8	3984	3862.6	121.4	60.7

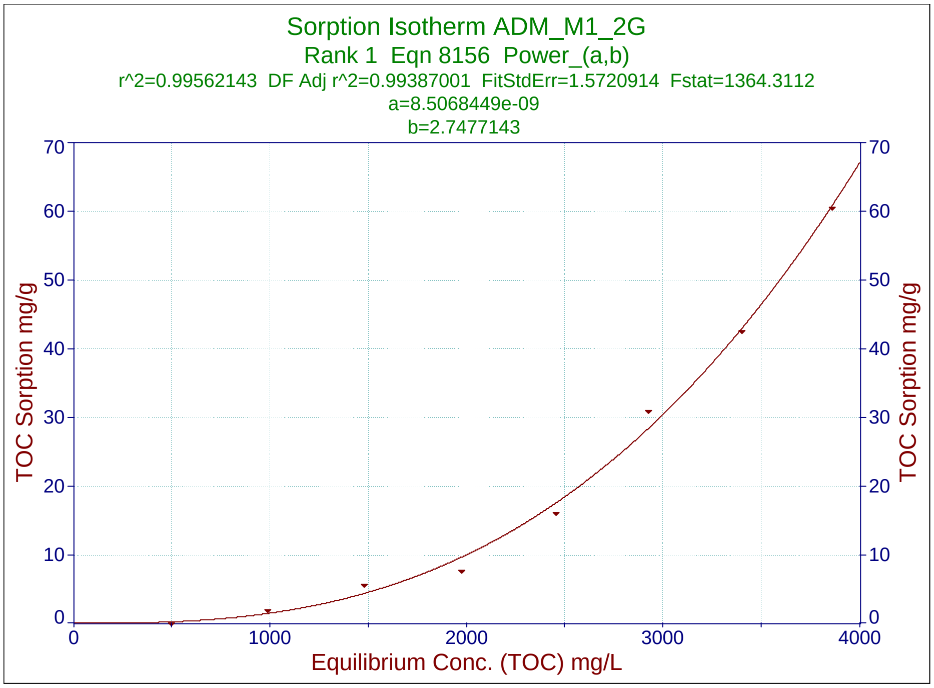


Fig. 6.13 Sorption Isotherm (Admixture M1_2g)

Table 6.15 Sorption Data for Admixture M1(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	497.5	0.5	0.2
2	996	990.5	5.5	1.8
3	1494	1481.2	12.8	4.3
4	1992	1961.8	30.2	10.0
5	2490	2431.4	58.6	19.5
6	2988	2919.8	68.2	22.7
7	3486	3357.6	128.4	42.8
8	3984	3798.7	185.3	61.7

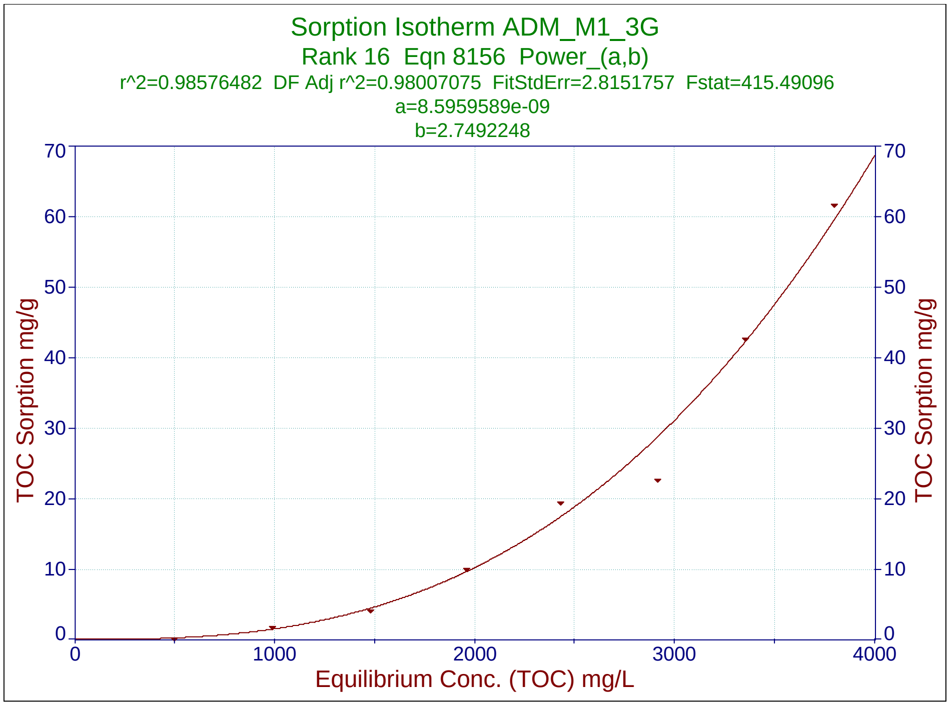


Fig. 6.14 Sorption Isotherm (Admixture M1_3g)

Table 6.16 Sorption Data for Admixture M 2(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	994.5	1.5	1.5
3	1494	1491.6	2.4	2.4
4	1992	1988.5	3.5	3.5
5	2490	2484.1	5.9	5.9
6	2988	2978.5	9.5	9.5
7	3486	3471.4	14.6	14.6
8	3984	3961.9	22.1	22.1

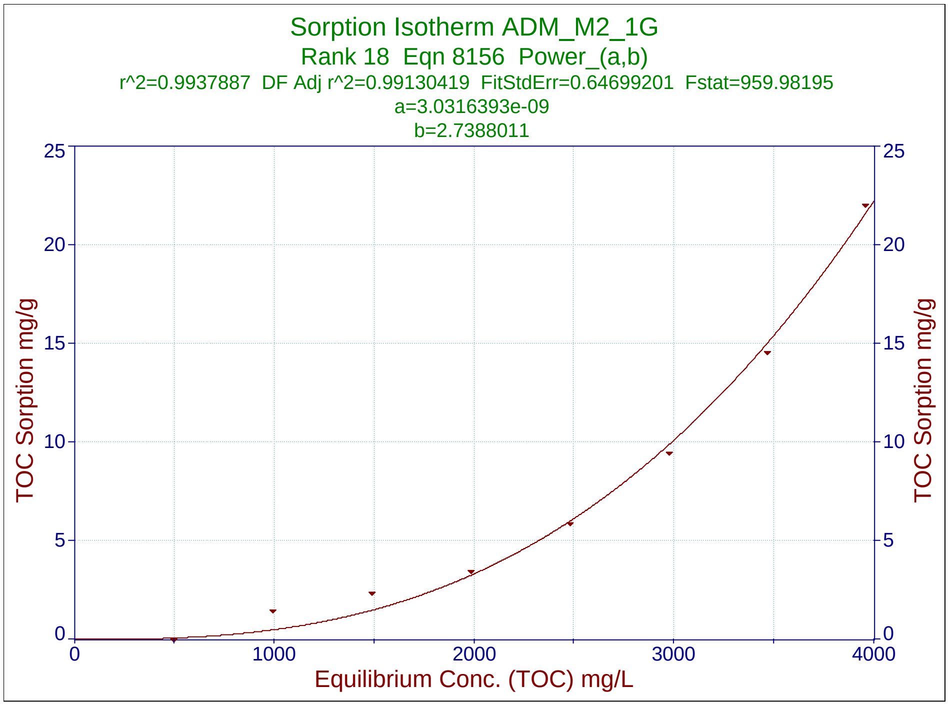


Fig. 6.15 Sorption Isotherm (Admixture M2_1g)

Table 6.17 Sorption Data for Admixture M 2(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	993.4	2.6	1.3
3	1494	1489.5	4.5	2.2
4	1992	1985.5	6.5	3.2
5	2490	2479.8	10.2	5.1
6	2988	2969.6	18.4	9.2
7	3486	3454.8	31.2	15.6
8	3984	3939.5	44.5	22.2

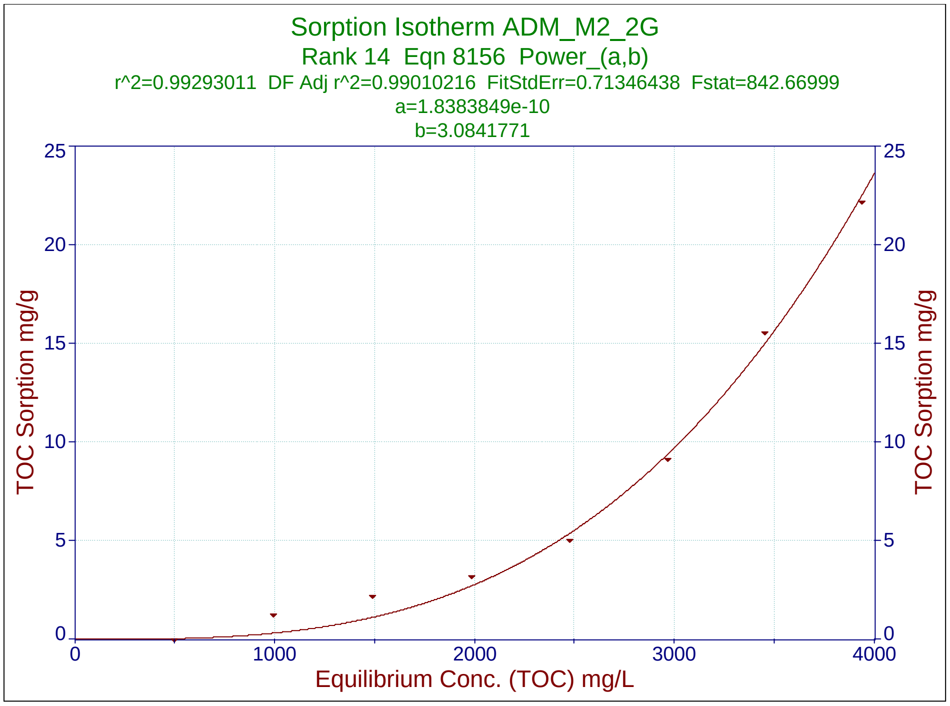


Fig. 6.16 Sorption Isotherm (Admixture M2_2g)

Table 6.18 Sorption Data for Admixture M 2(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	994.5	1.5	0.5
3	1494	1490.4	3.6	1.2
4	1992	1983.8	8.2	2.7
5	2490	2472.7	17.3	5.8
6	2988	2959.4	28.6	9.5
7	3486	3441.3	44.7	14.9
8	3984	3918.4	65.6	21.8

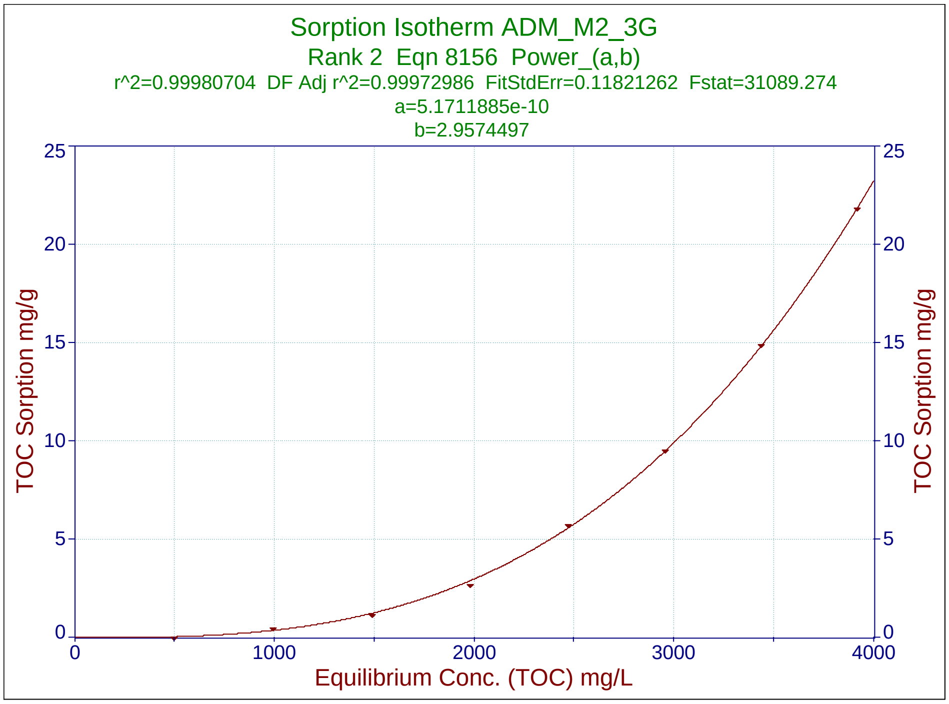


Fig. 6.17 Sorption Isotherm (Admixture M2_3g)

Table 6.19 Sorption Data for Admixture M 3(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	994.6	1.4	1.4
3	1494	1492.2	1.8	1.8
4	1992	1989.6	2.4	2.4
5	2490	2486.9	3.1	3.1
6	2988	2983.2	4.8	4.8
7	3486	3478.8	7.2	7.2
8	3984	3973.4	10.6	10.6

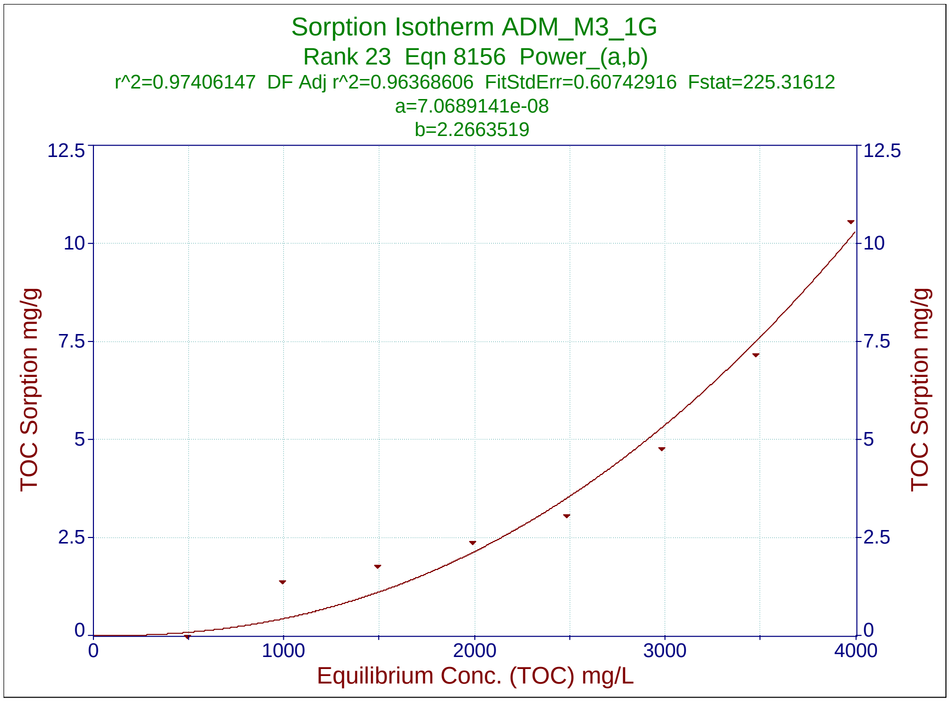


Fig. 6.18 Sorption Isotherm (Admixture M3_1g)

Table 6.20 Sorption Data for Admixture M 3(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	993.9	2.1	1.0
3	1494	1491.1	2.9	1.4
4	1992	1987.5	4.5	2.2
5	2490	2483.9	6.1	3.0
6	2988	2978.6	9.4	4.7
7	3486	3471.5	14.5	7.2
8	4000	3961.6	22.4	11.2

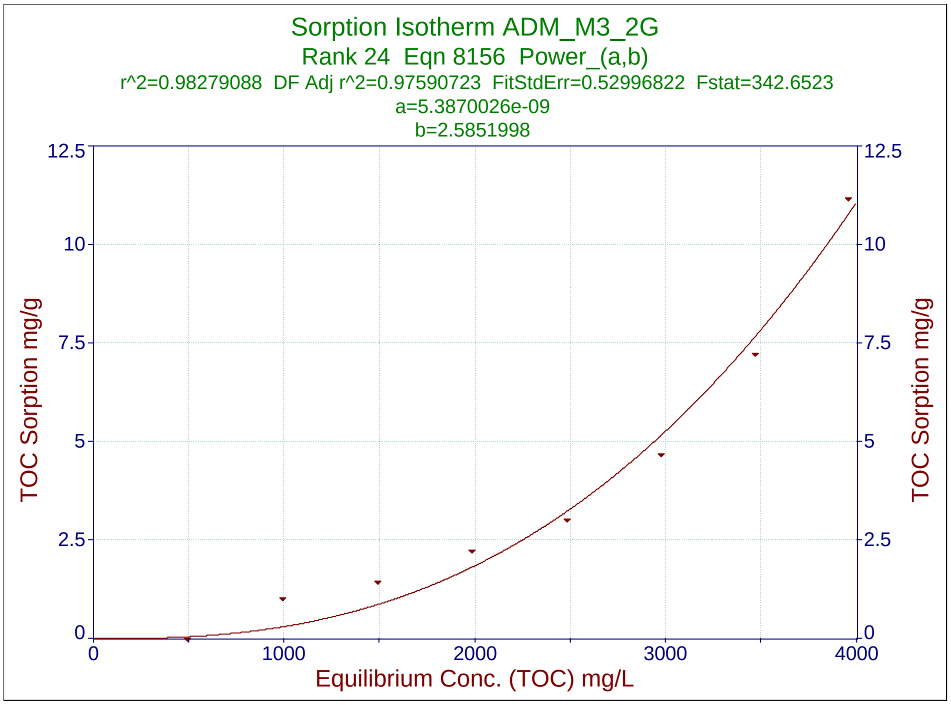


Fig. 6.19 Sorption Isotherm (Admixture M3_2g)

Table 6.21 Sorption Data for Admixture M 3(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	992.5	3.5	1.2
3	1494	1489.4	4.6	1.5
4	1992	1985.2	6.8	2.3
5	2490	2481.3	8.7	2.9
6	2988	2974.2	13.8	4.6
7	3486	3465.8	20.2	6.7
8	3984	3954.4	29.6	9.9

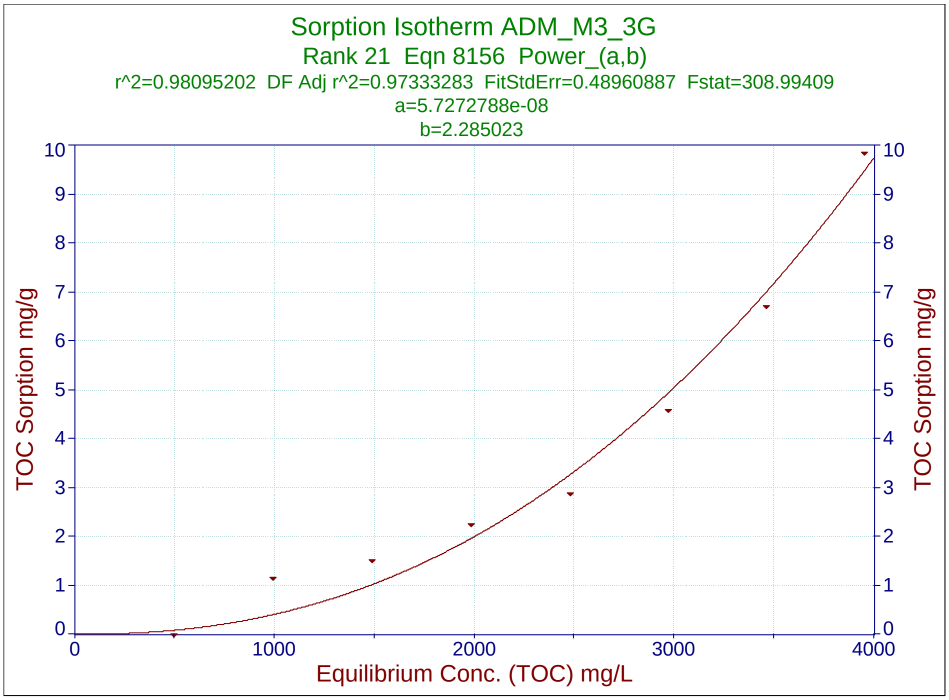


Fig. 6.20 Sorption Isotherm (Admixture M3_3g)

Table 6.22 Sorption Data for Admixture M 4(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	994.9	1.1	1.1
3	1494	1492.6	1.4	1.4
4	1992	1988.2	3.8	3.8
5	2490	2483.4	6.6	6.6
6	2988	2976.8	11.2	11.2
7	3486	3468.4	17.6	17.6
8	3984	3958.1	25.9	25.9

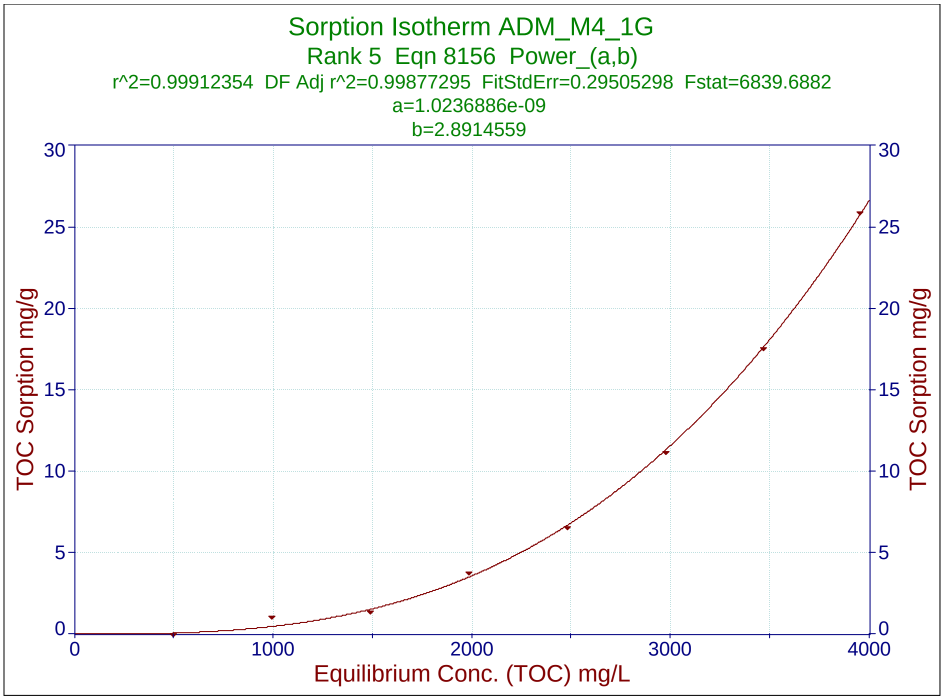


Fig. 6.21 Sorption Isotherm (Admixture M4_1g)

Table 6.23 Sorption Data for Admixture M 4(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	498	0	0
2	996	994.1	1.9	0.9
3	1494	1490.8	3.2	1.6
4	1992	1984.6	7.4	3.7
5	2490	2476.5	13.5	6.7
6	2988	2965.5	22.5	11.2
7	3486	3451.2	34.8	17.4
8	3984	3931.8	52.2	26.1

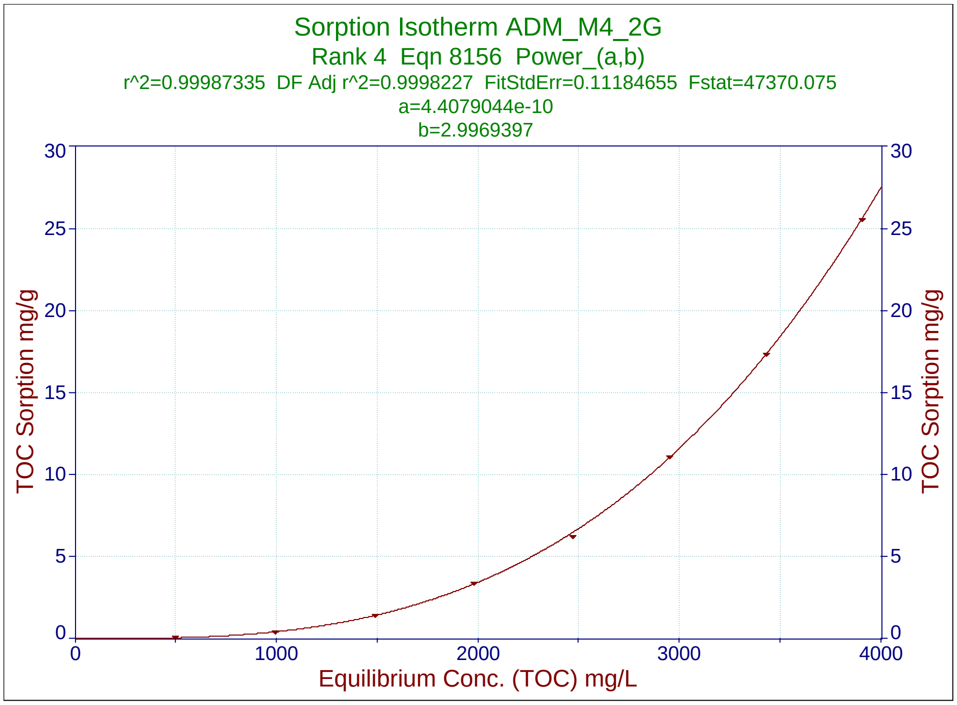


Fig. 6.22 Sorption Isotherm (Admixture M4_2g)

Table 6.24 Sorption Data for Admixture M 4(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	497.5	0.5	0.2
2	996	994.6	1.4	0.5
3	1494	1489.5	4.5	1.5
4	1992	1981.6	10.4	3.5
5	2490	2471.1	18.9	6.3
6	2988	2954.6	33.4	11.1
7	3486	3433.7	52.3	17.4
8	3984	3907.1	76.9	25.6

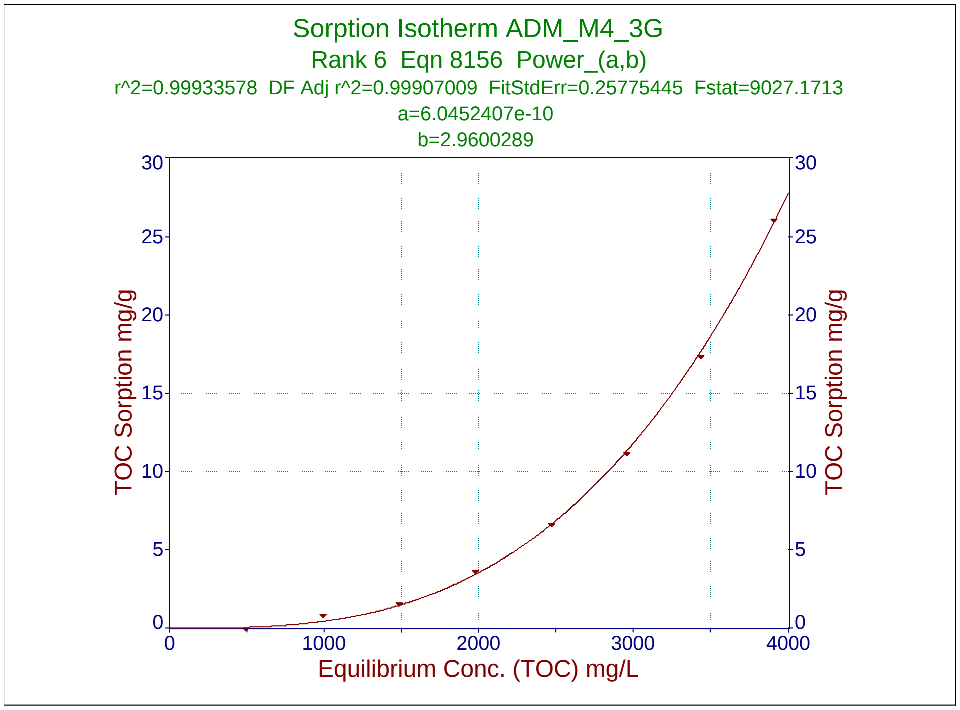


Fig. 6.23 Sorption Isotherm (Admixture M4_3g)

Table 6.25 Sorption Data for Admixture M 5(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	497.7	0.3	0.3
2	996	994.2	1.8	1.8
3	1494	1490.8	3.2	3.2
4	1992	1984.2	7.8	7.8
5	2490	2476.9	13.1	13.1
6	2988	2965.5	22.5	22.5
7	3486	3451.5	34.5	34.5
8	3984	3934.7	49.3	49.3

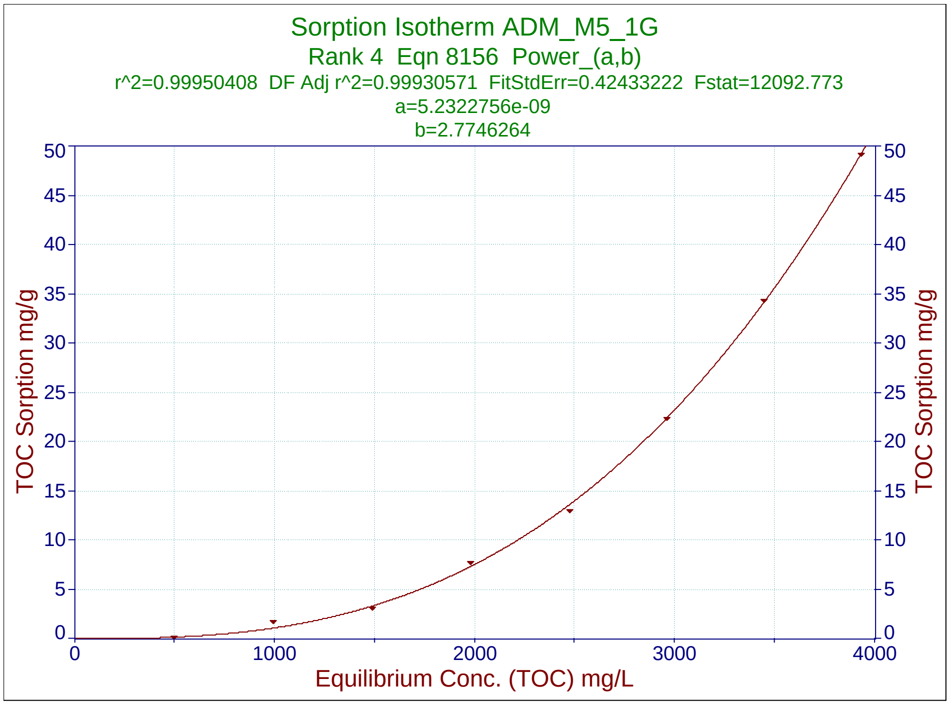


Fig. 6.24 Sorption Isotherm (Admixture M5_1g)

Table 6.26 Sorption Data for Admixture M 5(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	497.1	0.9	0.4
2	996	992.6	3.4	1.7
3	1494	1486.5	7.5	3.7
4	1992	1976.5	15.5	7.7
5	2490	2463.4	26.6	13.3
6	2988	2943.2	44.8	22.4
7	3486	3417.1	68.9	34.4
8	3984	3885.7	98.3	49.1

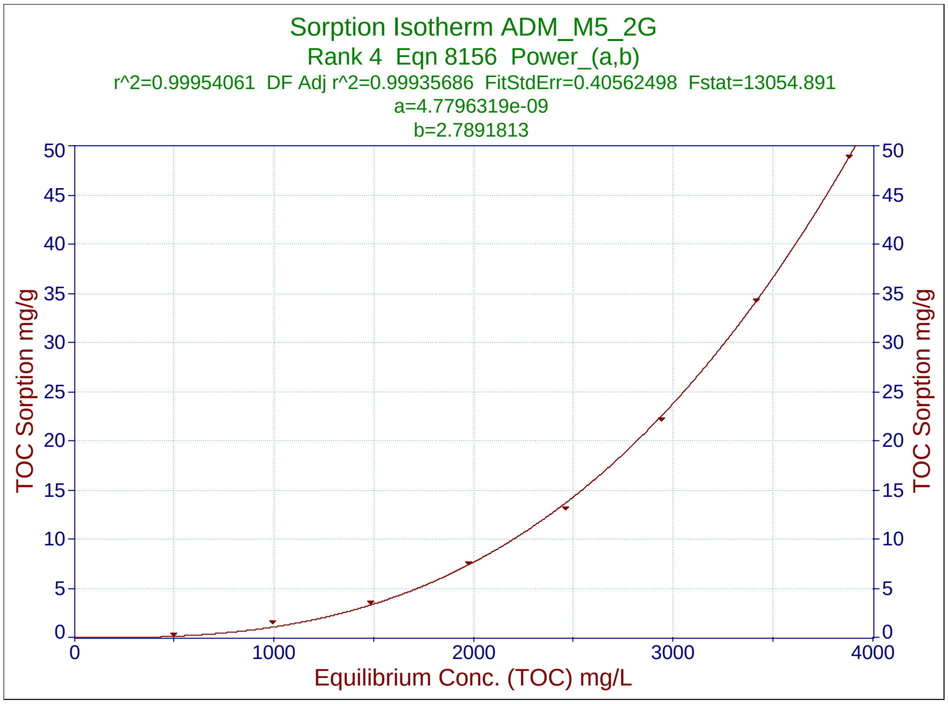


Fig. 6.25 Sorption isotherm (Admixture M5_2g)

Table 6.27 Sorption Data for Admixture M 5(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	498	496.9	1.1	0.4
2	996	992.4	3.6	1.2
3	1494	1485.4	8.6	2.9
4	1992	1970.6	21.4	7.1
5	2490	2448.8	41.2	13.7
6	2988	2921.5	66.5	22.2
7	3486	3384.2	101.8	33.9
8	3984	3837.7	146.3	48.8

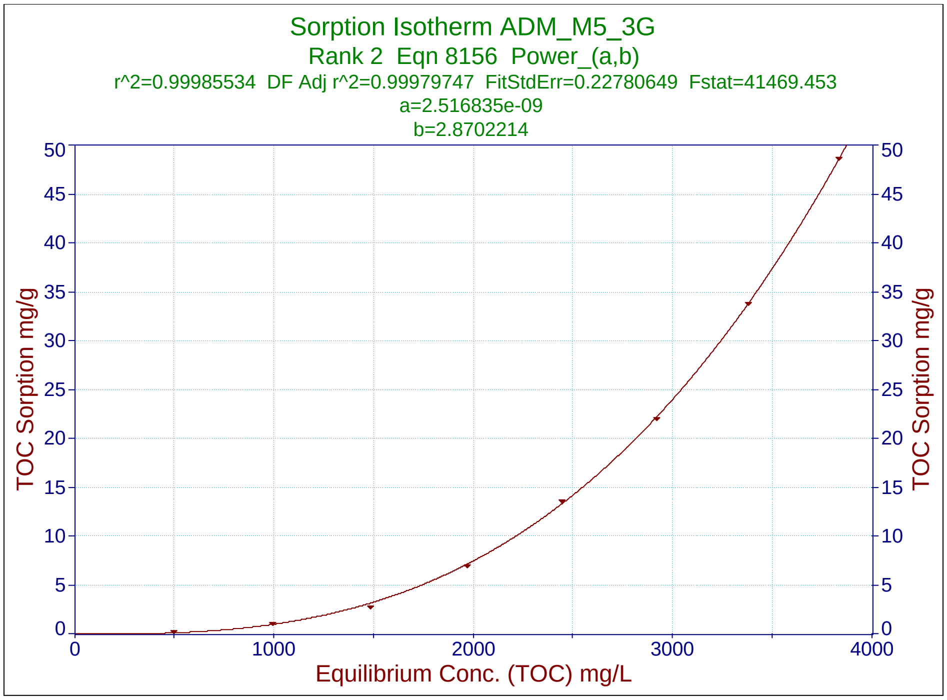


Fig. 6.26 Sorption Isotherm (Admixture M5_3g)

Table 6.28 Sorption Data for Admixture MC_ 1(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	500	499.2	0.8
2	1000	1000	997.4	2.6
3	1500	1500	1493.9	6.1
4	2000	2000	1991.8	8.2
5	2500	2500	2482.1	17.9
6	3000	3000	2965.4	34.6
7	3500	3500	3451.1	48.9
8	4000	4000	3934.2	65.8

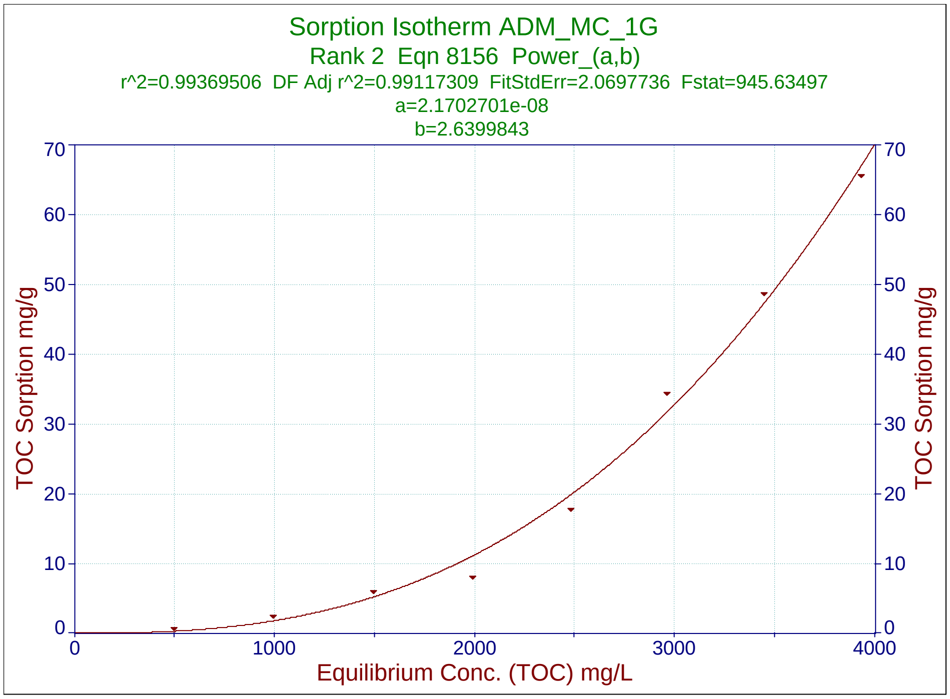


Fig. 6.27 Sorption Isotherm (Admixture MC_1_1g)

Table 6.29 Sorption Data for Admixture MC_ 1(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	498.8	1.2	0.6
2	1000	995.5	4.5	2.2
3	1500	1488.8	11.2	5.6
4	2000	1983.3	16.7	8.3
5	2500	2463.8	36.2	18.1
6	3000	2935.5	64.5	32.2
7	3500	3410.8	89.2	44.6
8	4000	3873.4	126.6	63.3

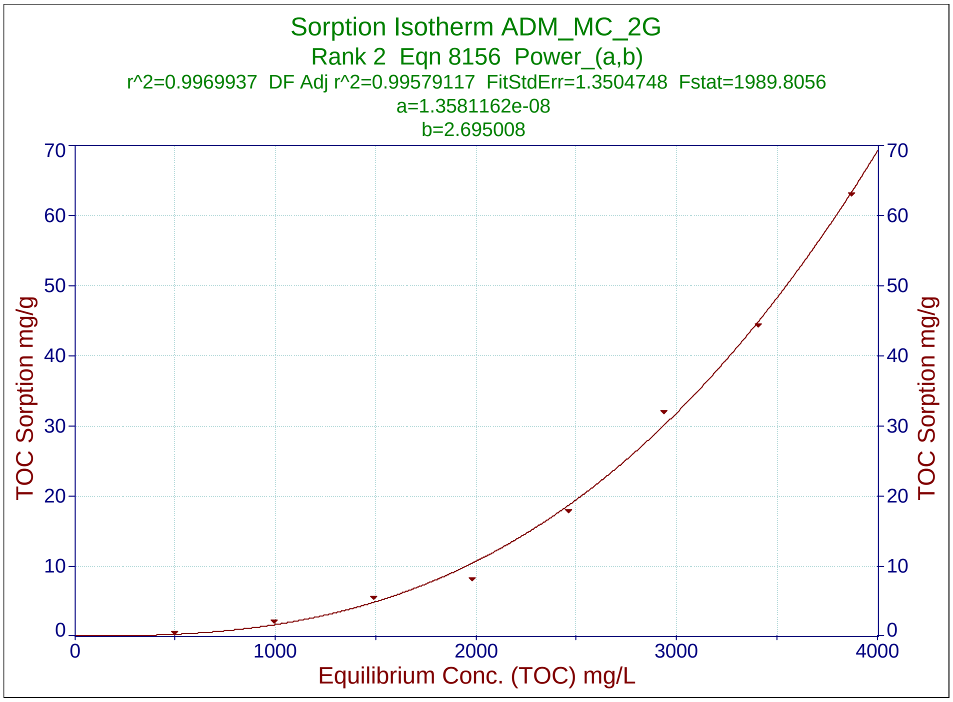


Fig. 6.28 Sorption Isotherm (Admixture MC_1_2g)

Table 6.30 Sorption Data for Admixture MC_ 1(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	500	0	0.5
2	1000	993.7	6.3	2.4
3	1500	1483.8	16.2	5.9
4	2000	1977.5	22.5	8.0
5	2500	2449.6	50.4	18.2
6	3000	2904.9	95.1	32.8
7	3500	3369.5	130.5	45.2
8	4000	3814.3	185.7	64.7

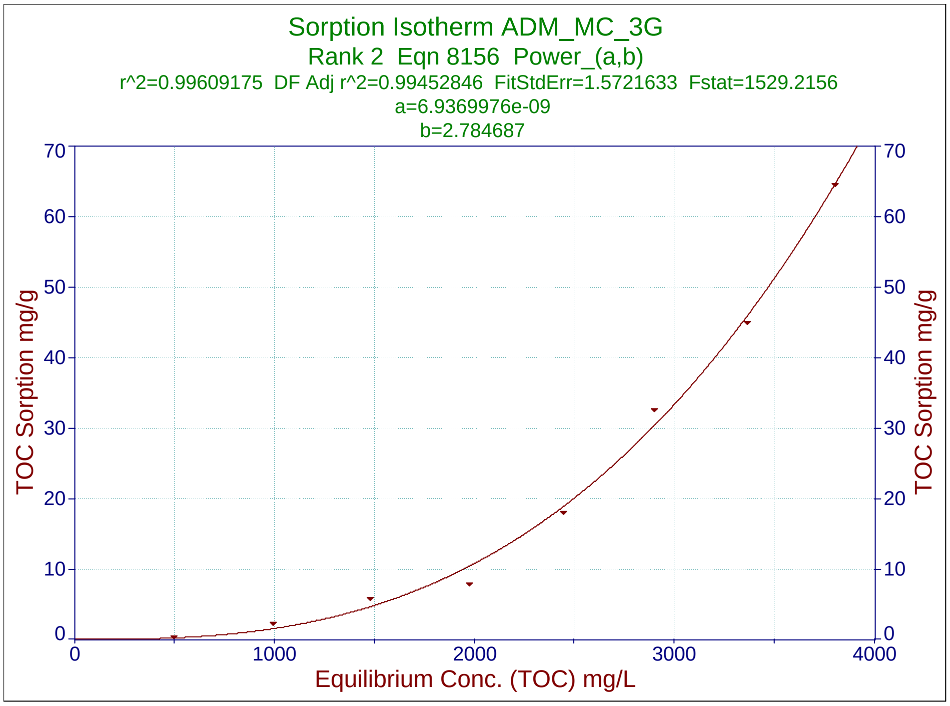


Fig. 6.29 Sorption Isotherm (Admixture MC_1_3g)

Table 6.31 Sorption Data for Admixture MC_ 2(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	499.4	0.6	0.6
2	1000	997.8	2.2	2.2
3	1500	1495.3	4.7	4.7
4	2000	1993.4	6.6	6.6
5	2500	2485.4	14.6	14.6
6	3000	2974.7	25.3	25.3
7	3500	3461.6	38.4	38.4
8	4000	3948.2	51.8	51.8

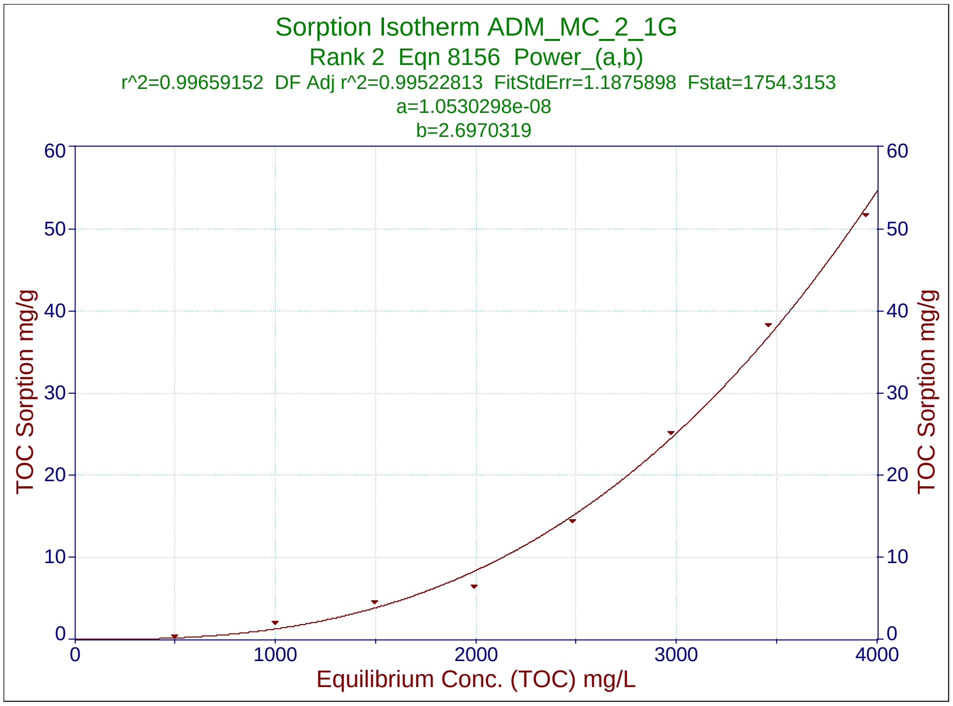


Fig. 6.30 Sorption Isotherm (Admixture MC_2_1g)

Table 6.32 Sorption Data for Admixture MC_ 2(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	499.2	0.8	0.4
2	1000	996.1	3.9	1.9
3	1500	1492.4	7.6	3.8
4	2000	1988.6	11.4	5.7
5	2500	2473.4	26.6	13.3
6	3000	2950.2	49.8	24.9
7	3500	3431.3	68.7	34.3
8	4000	3900.9	99.1	49.5

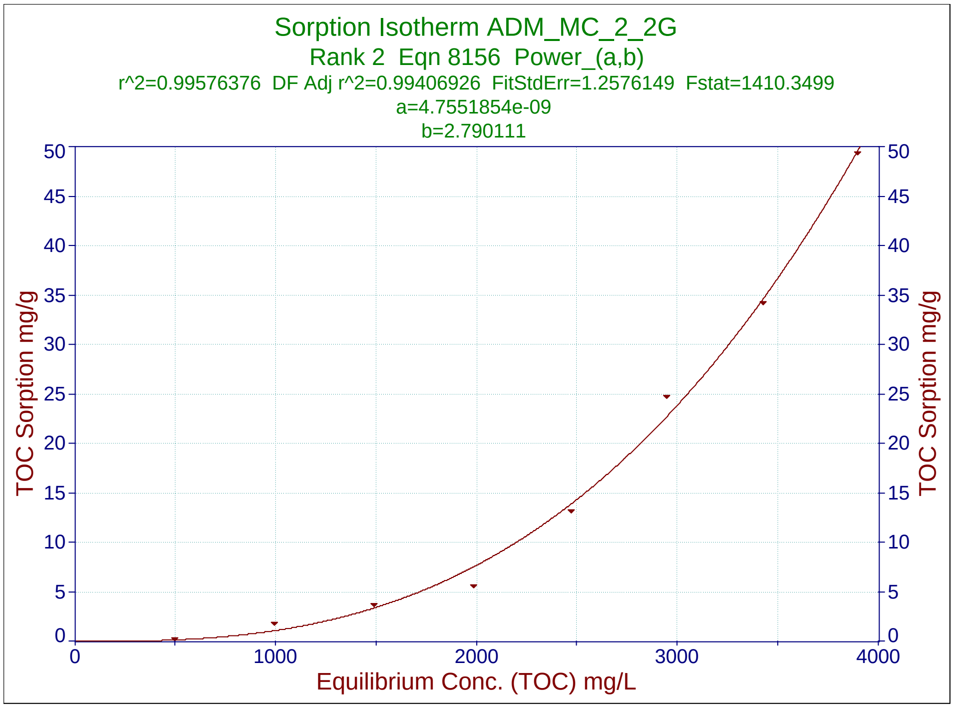


Fig. 6.31 Sorption Isotherm (Admixture MC_2_2g)

Table 6.33 Sorption Data for Admixture MC_ 2(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	499.1	0.9	0.3
2	1000	994.8	5.2	1.7
3	1500	1488.2	11.8	3.9
4	2000	1982.5	17.5	5.8
5	2500	2461.8	38.2	12.7
6	3000	2924.5	75.5	25.2
7	3500	3398.6	101.4	33.8
8	4000	3857.7	142.3	47.4

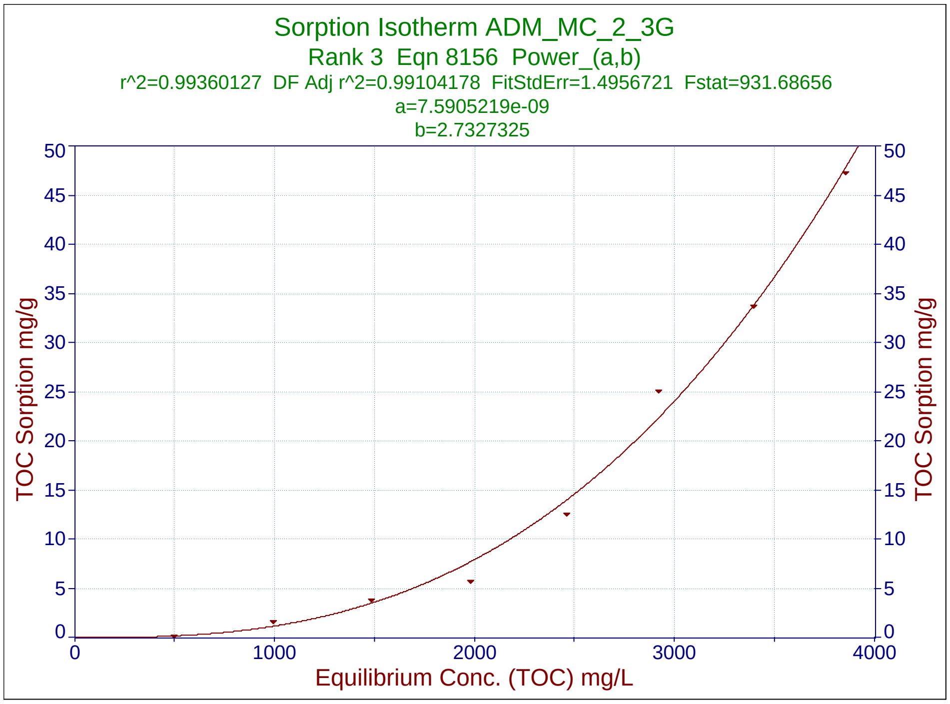


Fig. 6.32 Sorption Isotherm (Admixture MC_2_3g)

Table 6.34 Sorption Data for Admixture MC_ 3(1g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	500	0	0
2	1000	998.6	1.4	1.4
3	1500	1496.2	3.8	3.8
4	2000	1995.5	4.5	4.5
5	2500	2489.6	10.4	10.4
6	3000	2980.2	19.8	19.8
7	3500	3471.8	28.2	28.2
8	4000	3962.7	37.3	37.3

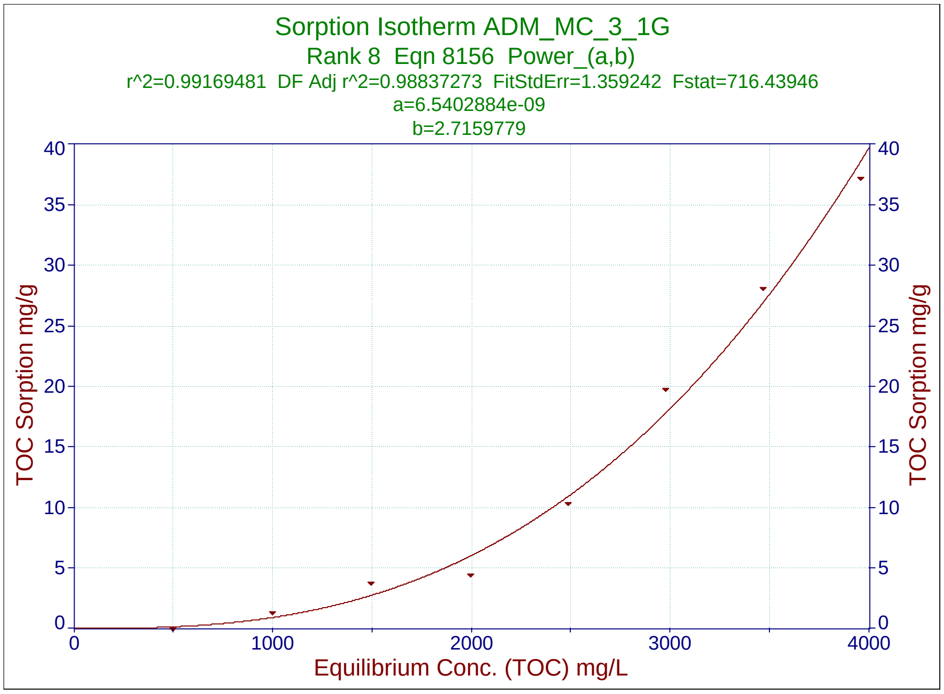


Fig. 6.33 Sorption Isotherm (Admixture MC_3_1g)

Table 6.35 Sorption Data for Admixture MC_ 3(2g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	499.5	0.5	0.2
2	1000	997.8	2.2	1.1
3	1500	1493.9	6.1	3.0
4	2000	1991.7	8.3	4.1
5	2500	2480.5	19.5	9.7
6	3000	2964.2	35.8	17.9
7	3500	3447.9	52.1	26.0
8	4000	3930.6	69.4	34.7

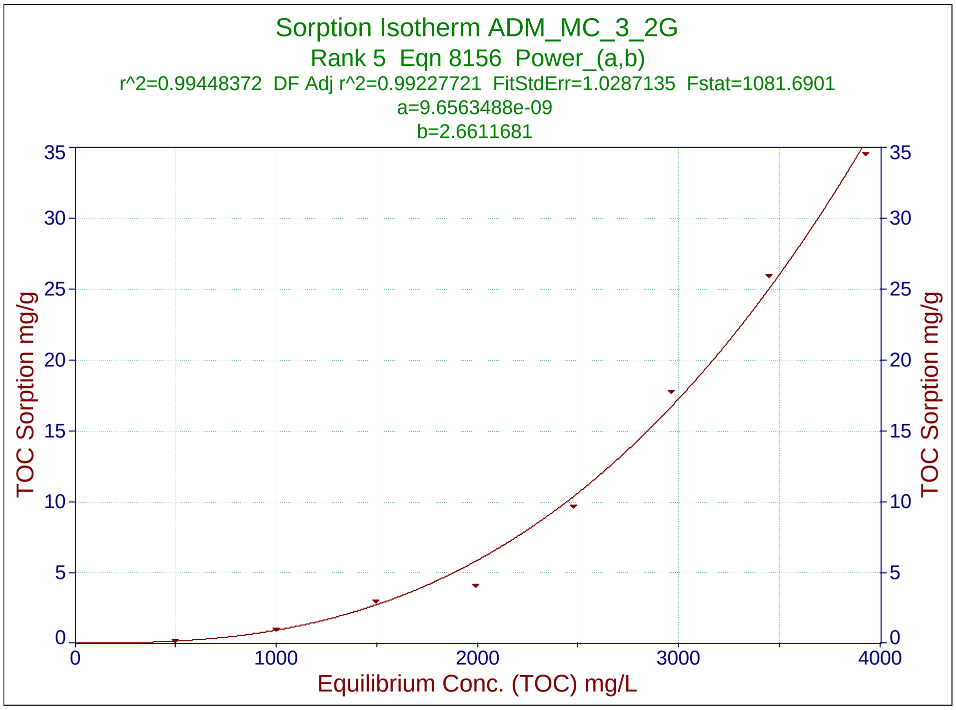


Fig. 6.34 Sorption Isotherm (Admixture MC_3_2g)

Table 6.36 Sorption Data for Admixture MC_ 3(3g)

S.No.	Initial Conc. TOC (mg/L)	Equilibrium Conc. TOC (mg/L)	TOC Sorption (mg)	TOC Sorption (mg/g)
1	500	499.2	0.8	0.3
2	1000	996.2	3.8	1.3
3	1500	1490.5	9.5	3.2
4	2000	1987.2	12.8	4.3
5	2500	2471.1	28.9	9.6
6	3000	2945.2	54.8	18.3
7	3500	3425.1	74.9	24.9
8	4000	3894.3	105.7	35.2

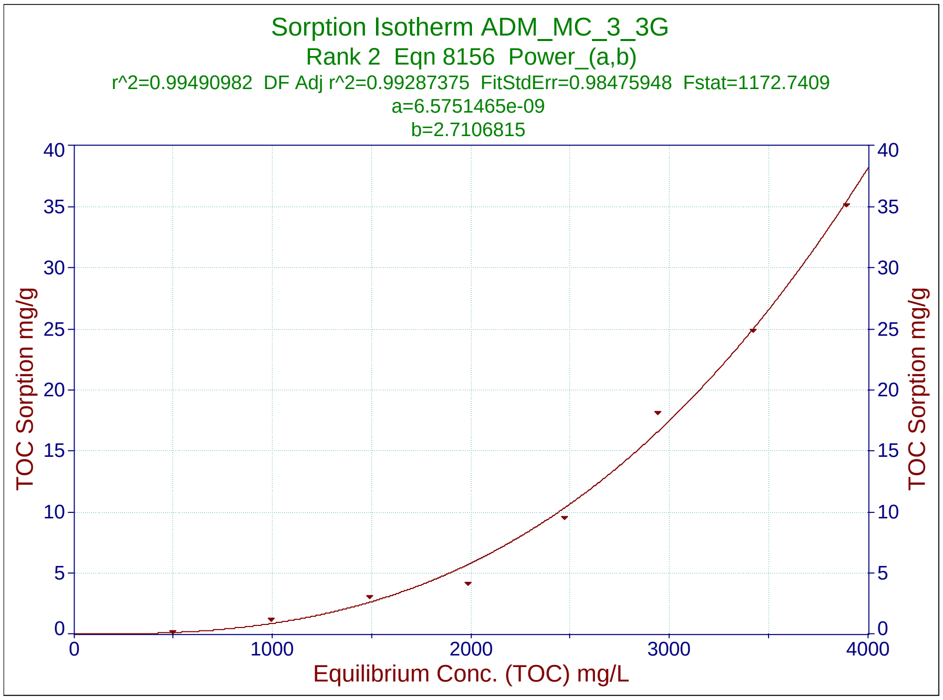


Fig. 6.35 Sorption Isotherm (Admixture MC_3_3g)

Table 6.37 Diffusion Coefficient Table (TOC)

S.No.	Admixture	ΔQ	ΔQ/Δt	*n*	C_0 (mg/L)	D*
1	M1	0.1582	9.15E-07	0.44	4980	2.50E-06
2	M2	0.2084	1.20E-06	0.45	4980	3.23E-06
3	M3	0.2592	1.50E-06	0.46	4980	3.92E-06
4	M4	0.2015	1.16E-06	0.47	4980	2.98E-06
5	M5	0.1856	1.07E-06	0.47	4980	2.75E-06
6	MC_1	0.2184	1.26E-06	0.44	4980	3.46E-06
7	MC_2	0.2015	1.16E-06	0.45	4980	3.12E-06
8	MC_3	0.1900	1.10E-06	0.46	4980	2.88E-06

Table 6.38 Diffusion Coefficient Table (Chloride)

S.No.	Admixture	ΔQ	ΔQ/Δt	*n*	C_0 (mg/L)	D*
1	M1	0.4712380	2.727E-06	0.44	6840	5.436E-06
2	M2	0.3771894	2.182E-06	0.45	6840	4.254E-06
3	M3	0.3139928	1.817E-06	0.46	6840	3.465E-06
4	M4	0.2632364	1.523E-06	0.47	6840	2.843E-06
5	M5	0.2453224	1.419E-06	0.47	6840	2.649E-06
6	MC_1	0.3368829	1.949E-06	0.44	6840	3.886E-06
7	MC_2	0.3139928	1.817E-06	0.45	6840	3.542E-06
8	MC_3	0.2945859	1.704E-06	0.46	6840	3.250E-06

6.5 Sorption Isotherms and Retardation Factor

TOC sorption for the soil admixtures were studied using batch sorption tests. The tests were carried out on leachate or standard TOC solution, and at room temperatures. Fig.12 to Fig. 6.26 shows the sorption isotherms of the TOC from the leachate on five types of admixtures prepared using synthesized organoclay. Similarly Fig.27 to Fig. 6.36 shows the sorption isotherms of the TOC from the standard TC solution on three types of admixtures prepared using manufactured organoclay. All the sorption curves can be seen to be having initial convexity and a final upsweep. Such sorption behavior is indicative of weak sorbent-sorbate interaction at low concentrations, followed by sorption of more molecules in a cooperative fashion, once a molecule is sorbed. It can be observed from these figures that the TOC sorption follows a pattern dependent on the composition of soil admixture. The soil admixture having a higher content of organoclay has higher sorption capacity for TOC. The admixture M1 which is composed of 25% organoclay and 75% natural soil has maximum sorption capacity for TOC, and the admixture M3 which is composed of 75% natural soil and 15%, 10% organoclay has minimum sorption capacity for TOC.

Similarly it can be seen that, for the case of admixtures prepared using manufactured organoclays, maximum retardation is for the admixture which has highest percentage of organoclay content, and least retardation factor for the admixture with lowest organoclay content.

Since the sorption isotherms are nonlinear, the partition coefficient, K_f, is not constant but is a function of the equilibrium concentration of TOC. The variation of partition coefficient with equilibrium concentration depends on the isotherm shape. The slope of a convex isotherm at a lower concentration is smaller than that at a higher concentration. It is difficult to make use of a non uniform partition coefficient in a one-dimensional contaminant transport model. An approximation of a representative linear partition coefficient is therefore essential based on weight-mean method. With the weighted-mean method, the average value of the partition coefficient, K_f and the retardation factor, R, are estimated using following equations:

$$K_p = \frac{\int_0^c \frac{\partial S}{\partial S} dC}{\int_0^c dC} = \frac{\int_0^c K_f C^{b-1} dC}{\int_0^c dC} = K_f C^{b-1} \qquad \text{...........6.4}$$

$$R = 1 + \frac{\rho K_f C^{b-1}}{n} \qquad \text{..............6.5}$$

Where ρ is bulk density of the soil in kg/m^3 and n is soil porosity. The retardation factors calculated at C=0.5C_0 for all the admixtures is shown in Table 6.39. The reason for using C=0.5C_0 is because it is more representative of the equilibrium concentration of TOC. The physical meaning of R in this case can be defined as the ratio of the time required for a contaminant to attain a half-maximum concentration as compared with that for a conservative tracer.

Table 6.39 Sorption Isotherms

Admixture	W=1g		W= 2g		W= 3g	
	Freundlich Isotherm	R	Freundlich Isotherm	R	Freundlich Isotherm	R
M1	$1.19 \times 10^{-8} C^{2.703}$	21.41	$8.50 \times 10^{-9} C^{2.747}$	21.57	$8.59 \times 10^{-9} C^{2.749}$	22.11
M2	$3.03 \times 10^{-9} C^{2.738}$	7.51	$1.83 \times 10^{-10} C^{3.084}$	6.88	$5.17 \times 10^{-10} C^{2.957}$	7.15
M3	$7.07 \times 10^{-8} C^{2.266}$	4.54	$5.38 \times 10^{-9} C^{2.585}$	4.27	$5.72 \times 10^{-8} C^{2.285}$	4.33
M4	$1.02 \times 10^{-9} C^{2.891}$	7.42	$4.40 \times 10^{-10} C^{2.996}$	7.29	$6.04 \times 10^{-10} C^{2.960}$	7.52
M5	$5.23 \times 10^{-9} C^{2.774}$	14.00	$4.77 \times 10^{-9} C^{2.789}$	14.34	$2.51 \times 10^{-9} C^{2.870}$	14.22
MC_1	$2.17 \times 10^{-9} C^{2.640}$	22.80	$1.35 \times 10^{-8} C^{2.695}$	21.86	$6.93 \times 10^{-9} C^{2.784}$	22.48
MC_2	$1.05 \times 10^{-9} C^{2.697}$	17.37	$4.75 \times 10^{-9} C^{2.790}$	16.33	$7.59 \times 10^{-9} C^{2.732}$	16.56
MC_3	$6.54 \times 10^{-9} C^{2.715}$	12.39	$9.65 \times 10^{-9} C^{2.661}$	12.01	$6.57 \times 10^{-9} C^{2.710}$	12.00

6.6 Transport Modeling Simulations

The experimental results from the sorption and permeability experiments were used as input data for transport simulations in order to evaluate the potential benefit of using sorptive amendments as a component of compacted clay liners. The transport simulations for the organic contaminants represented by TOC were performed using the contaminant transport model developed in this study. The model is represented by Eq. 3.10, and subjected to finite mass boundary condition 3.17. The retardation factor used for transport modeling simulations was one determined in this study for different types of admixtures using sorption isotherms (Table 6.39).

Transport simulations were carried out using values of v_s, and R and D_h determined in this study. Total time period of simulation was taken as 50 years accounting for the first 10–15 years for waste receiving at landfill site and the remaining for post-closure. Eq. (3.10) representing transport of contaminants was solved using finite difference method implemented in MatLab 7.0. Simulation results were used for determining the maximum relative concentration at any depth within the liner and plotting the same. Simulations were carried for five types of admixtures. The results of simulation plotted in the form of graphs between maximum relative concentrations with respect to depth from the top of liner can be used as design chart. Such chart can be utilized for the determination of liner thickness if the maximum permissible relative concentration after a certain period of time is defined based on pollution prevention criteria in a region. Fig. 6.36 and Fig. 6.37 presents maximum relative concentration for a traveling time of 50 years for different admixtures. It is obvious that the required thickness is highly dependent on the retardation factor and hydraulic conductivity of the specimen. The required thickness for the admixture M1 is the least, as it has highest retardation factor. Similarly the required thickness for admixture M3 is largest since it has the lowest retardation factor. Therefore, the only way to reduce the chemical flux is to improve the retardation factor of the soil material. The rule of thumb is that the larger the retardation factor of a liner material is, the lesser the liner thickness. This, in turn, saves more space for landfill disposal of solid wastes.

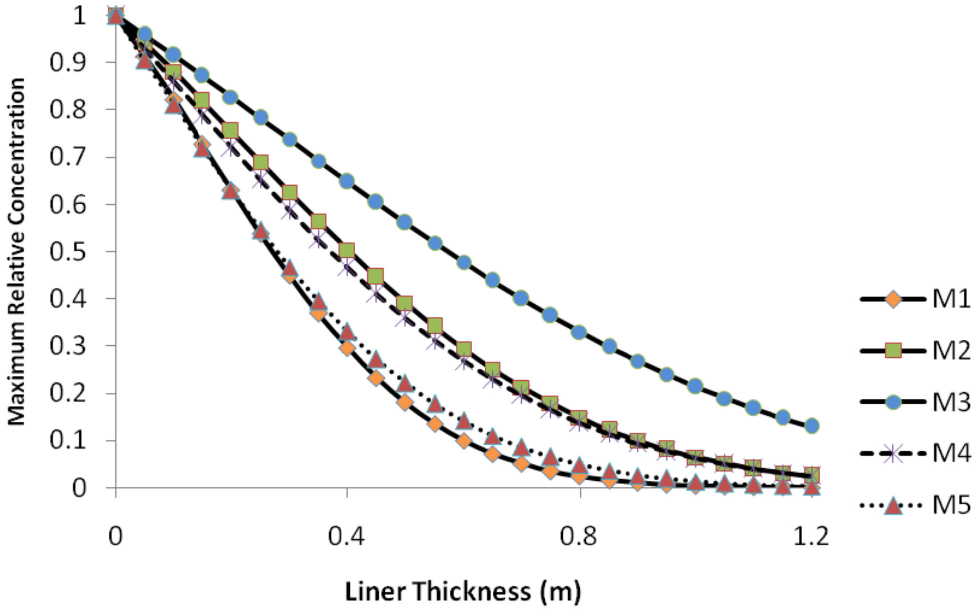


Fig. 6.36 Comparison of Amended Liners (synthesized organoclay) Time = 50 years (TOC)

6.7 Conclusions

In this study, both experimental work and simulation modeling were conducted. Experimental data on various soil admixtures prepared by varying the content of organoclay and bentonite in natural soil show that replacing 15% of bentonite with synthesized organoclay increases the retardation factor from 4.54 to 21.41. However, the presence of bentonite in soil specimen is also necessary, as it could reduce hydraulic conductivity of admixtures. A design chart was developed by the solving 1D contaminant transport mass transport equation using finite difference technique. The chart provides a logical method for estimating the required thickness of a liner having composition based on these admixtures. The liner material with higher proportion of organoclay significantly reduces the required thickness because of the high sorption capacity. Similar trends were observed with the studies carried out on organoclay as manufacture product. It is obvious that the required thickness is highly dependent on the retardation factor and hydraulic conductivity of the specimen. The required thickness for the admixture M1 is the least, as it has highest retardation factor. Similarly the required thickness for admixture M3 is largest since it has the lowest retardation factor. Similar trends can be seen to be following for the admixtures prepared using manufactured organoclay in Fig. 6.37. Therefore, the only way to reduce the chemical flux is to improve the retardation factor of the soil material. The rule of thumb is that the larger the retardation factor of a liner material is, the lesser the liner thickness. This, in turn, saves more space for landfill disposal of solid wastes.

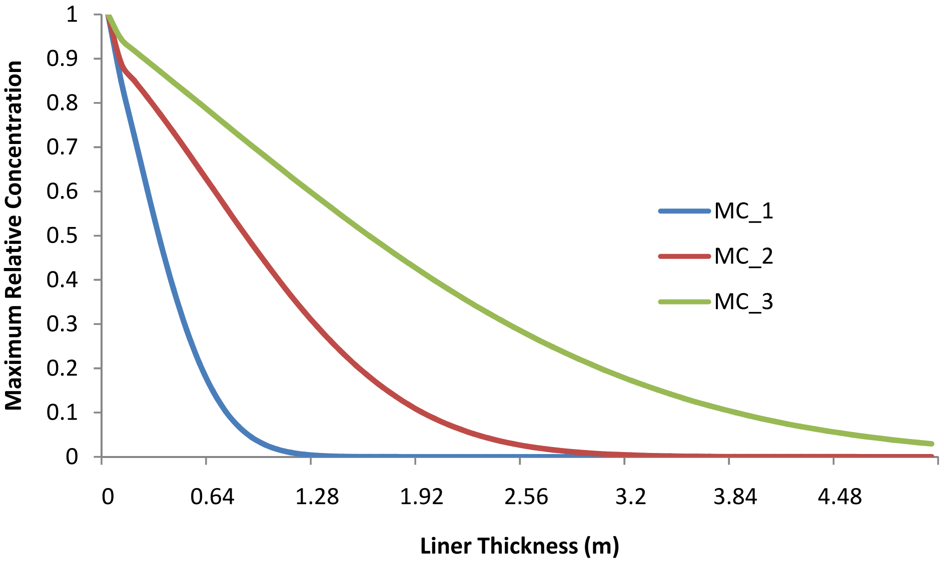


Fig. 6.37 Comparison of amended liners (Manufactured Organoclays) Time 50 years TOC

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SYNOPSIS

ON

CONTAMINANT TRANSPORT FROM LANDFILLS

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CONTENTS

ABSTRACT

Point sources such as landfills, can release high concentrations of contaminants into the groundwater because of migration of leachate from its bottom, which is generated primarily as a result of precipitation falling on an active landfill surface, leaching out the potential organic and inorganic contaminants from landfilled waste and discharging the same to groundwater in underlying aquifer. To protect the groundwater from contamination, it is quite essential to provide the bottom barrier of suitable thickness.

The present study was undertaken to determine the rate of movement of potential contaminants from the bottom of a landfill, so as to evolve a rational method for the determination of thickness of landfill liner. A mathematical model was formulated to express the mass transport of contaminants from a landfill due to the migration of leachate. Various mechanisms of contaminant migration from the bottom of landfill were taken into account. The solution of the model in the form of concentration profile of potential contaminants below landfill was obtained using the explicit Finite Difference Method implemented in MatLab 7.0. The model developed was validated for two cases of field data. The simulations of model were run for a range of permeability values, equivalent height of leachate, and retardation factors. The results of such simulations in the form of maximum relative concentration at various depths, at the end of design period of landfill were used for preparing the design charts. The design charts so prepared can be used for determination of minimum thickness of landfill liner on the basis of maximum permissible relative concentration of contaminant species of interest at the bottom of landfill liner. Design charts were prepared for both conservative and non conservative contaminant.

The contaminant transport model developed in this study was applied for determining the impact of Bhalaswa landfill at Delhi on the groundwater in its vicinity. For determining the impact of landfill on groundwater of the region, simulations of the model were run for tracer contaminant Chloride, and the results of such simulations were compared with the observed values of Chloride

concentration in the groundwater of the region. Groundwater samples were collected at varying radial distances from the landfill and in the direction of groundwater flow and analyzed for key contaminants. The results of such sampling and analysis show gradually decreasing concentration of the contaminants away from the landfill. Gas generation from Bhalaswa landfill has been estimated using public domain software based on first order kinetics.

Experimental studies were carried out on eight admixtures (five prepared using synthesized organoclay and three prepared using manufactured organoclay with bentonite and natural soil as remaining constituents) for the determination of their hydraulic and sorption characteristics. The results of such studies have been used for transport modeling simulations to determine the effectiveness of each of these admixtures with regard to retardation of organic contaminants from landfill leachate and consequent impact on minimum thickness of liner. Design charts were drawn for the determination of minimum liner thickness based on organic contaminant, and using these admixtures. Retardation factor of the admixtures was found to increase with increase of organoclay content. The sorption behavior of all the admixtures have been found to be consistent and scalable.

CONTAMINANT TRANSPORT FROM LANDFILLS

1.0 Introduction

Groundwater is the largest source of freshwater on our planet, representing over 90% of the readily available freshwater reserves. This resource has two distinct functions: firstly, it is a significant source of both urban and rural water supply and secondly it sustains many wetland ecosystems. However, Groundwater can be polluted by landfills, septic tanks, the widespread use of road salts and chemicals, by leaky underground gas tanks, and also from overuse of fertilizers and pesticides. Pollutants can accumulate over years in the groundwater and suddenly become mobilized when the assimilative capacity of soil is exceeded, thus steadily diminishing the amount of clean water they can yield for human use. Ground-water quality is being increasingly threatened by agricultural, urban & industrial wastes which leach or are injected into underlying aquifers. It has been established that once pollution has entered the subsurface environment, it may remain concealed for many years, becoming dispersed over wide areas of ground water aquifer and rendering ground water supplies unsuitable for consumption and other uses. The rate of depletion of ground water levels and deterioration of ground water quality is of concern in major cities and towns of the country. Any contamination of groundwater would be a significant strain on the water distributing unit because rehabilitation of polluted groundwater is expensive, and often may not be technically feasible. Secondly, this means that water has to be abstracted from other sources and this would result in conflicts with other uses, especially if the land is facing a drought period, which is often the case. One way to protect aquifers is to control activities which pose a threat to groundwater quality, especially in areas where underground water is naturally poorly protected.

2.0 Motivation for Present Study

To comply with the pollution prevention criteria of the region the landfills need to be provided with the bottom barrier, thickness of which needs to be determined based on sound engineering principles which involves modeling of contaminant migration processes from landfill into the clayey barrier. The problem of migration of contaminants through natural barriers located below landfills has been addressed by many researchers in the past, but most of the methods developed so far have been analytical or semi analytical and each one worked under a set of limitations associated with their respective models. These models have limited application however some of these models are popular because they are simple and user friendly. The softwares available in public domain for the modeling of contaminant transport are not suitable for application to the problem of contaminant transport from landfills because of the special boundary conditions required. Moreover many of the softwares related to contaminant transport in porous media are neither available in public domain nor through any agency in India. There exists an absence of information and knowledge to face the challenges posed by solid disposal problem in Indian conditions with regard to barrier design where one finds a large variation in the quantity of wastes to be handled in varying hydrogeological conditions. There was thus, this gap which led to taking up this study for the development and validation of the mathematical model for transport of contaminants from the bottom of landfill, and consequent preparation of design charts on the basis of results of model simulations which can be readily used for the determination of minimum barrier thickness.

Further the modifications in the properties of landfill liner materials if needed can be achieved by adding certain special materials depending upon the function desired. Such materials are added to the landfill liner for getting enhanced retardation of target contaminants. Different materials may be added depending upon the type of contaminant and to the extent of desired retardation. Clays modified with cationic surfactant (organoclays) have been known to be having a considerable potential with regard to sorption and consequent retardation of organic compounds form landfill leachate. A study carried out by Lo (Lo 2003), have shown that the retardation factor of the landfill barrier against organic contaminants can be obtained as high as about 12.5 by amending such materials with 20% of organoclay. The enhancement of retardation factor will have direct bearing on the minimum thickness of landfill barrier required to comply with pollution prevention criteria in a region. A higher retardation factor shall lead to greater reduction in the thickness of the barrier required on the basis organic contaminants. No such studies have been undertaken in India so far and import of such amendments if required can run into huge costly affair, unthinkable for municipalities in India. It was thus thought of to take up a study involving the use of liner materials amended with locally available organoclay, for the determination of the impact of such amendments on the sorption and consequent retardation of organic contaminants in landfill leachate, using the leachate obtained from Bhalaswa landfill site in Delhi. The study was thus aimed at determining the impact of amending the barrier materials on retardation of organic contaminants from landfill leachate using locally available materials, and under Indian conditions.

3.0 Objectives of Study

Looking to the current practice of municipal solid waste disposal practice being followed in India and the extent of likely impact on groundwater contamination, a study was taken with the focus on identification of groundwater contamination potential of landfill and control measures for the prevention of such contamination. Following were the objectives of present study:

1. To develop and validate mathematical model that would address the problem of migration of contaminants from the bottom of landfill.
2. To apply the model developed in this study for the determination of likely impact of Bhalaswa landfill site at Delhi on the contamination of groundwater in its vicinity.

3. To prepare the design charts which may be used for the determination of minimum thickness of landfill liner for a typical hydrogeological conditions and pollution prevention criteria.
4. To study the various admixtures for landfill liners for the control of migration of contaminants from landfill leachate.

4.0 Scope of the Study

Scope of present study was limited to the development and validation of mathematic model to address the problem of contaminant migration from the bottom of landfill, taking into account the various mechanisms of contaminant migration in such a case. The case study of Bhalaswa landfill site was limited in the scope to the application of model to the landfill site for the purpose of model validation and determination of likely impact of landfill on the groundwater in the vicinity of landfill site. Preparation of design charts for the landfill liner thickness was limited to plotting of maximum relative concentration of key contaminants below landfill with respect to depth on the basis of simulation results of model. Studies on admixtures were limited to the determination of the model parameters for admixtures prepared in the present study and consequent evaluation of their effectiveness with regard to contaminant retardation.

5.0 Literature Review

A sanitary landfill is defined as a system in which municipal solid wastes are disposed of, compacted, and covered with a layer of soil at the end of each day’s operation. However, poorly designed landfills can create contamination of groundwater, soil, and air. A review of the literature highlights the research efforts directed towards various aspects of solid waste disposal by landfilling viz. quantity and quality of leachate, leachate migration from landfills, estimation of landfill gas emissions, landfill liner materails.

Tchobanoglous et al. (1997) studied the moisture content of solid wastes, and concluded that for most MSW the moisture content varies from 15 to 40 %, depending on humidity and weather conditions, waste composition, and the season of year.

Michaels and Lin (1954) studied the factors affecting the hydraulic conductivity of low permeability liner materials and concluded that the nature of permeant can significantly affect the hydraulic conductivity of soil. The large variations in permeability at a given void ratio can be explained by the differences in soil structure resulting from using different fluids during sample preparation. Hydraulic conductivity varies with the cube of degree of saturation, which means that the permeability should increase as the degree of saturation increases.

Determination of expected quantity of leachate generated is crucial to the design of Leachate Collection and Removal (LCRS) system. The total quantity of leachate generated at a given waste containment system is primarily a function of the quantity of water infiltrated into the system and quantity of fluids generated within the waste. The former is turn in dependent on a number and intensity of climatologic and hydrologic processes, primarily rainfall, runoff and evaporation. Thus, to estimate the quantity of leachate, one needs to conduct a water balance for the entire system (Fenn et al. 1975).

Leachate quantity is usually modeled and/or determined using a simple water balance approach taking into account the amounts of water entering the landfill (i.e. precipitation, waste moisture in excess of moisture holding capacity of the waste and additional water input such as water in wastewater treatment plant sludge if allowed) and the amounts of water leaving the landfill (i.e. water consumed in biochemical reactions and evaporation). The water balance method assumes that precipitation either runs off from the landfill or infiltrates the surface (uncovered refuse, intermediate cover, or final cover). Some of the infiltrated water evaporates from the surface and (or) transpires through the vegetative cover if it exists. Some of infiltration may make up a deficiency in soil moisture storage, the difference between field capacity (FC) and the existing moisture content (MC). The remainder of infiltration moves downward forming percolate (PERC) and eventually leachate (L) as it reaches the base of the landfill. PERC may be augmented by infiltration of groundwater

The water balance methods are based on procedures developed by Thornthwaite (Thornthwaite and Mather 1955, 1957; Thornthwaite 1964) in the soil and water conservation field. Since his work many research efforts have been directed to develop the water balance equations in the last 50 years (Fenn et al. 1975; Perrier and Gibson 1980; Knisel and Nicks 1980; Skaggs 1980; Schroeder et al. 1984a, 1984b). These water balance models consider the landfill a “black box,” requiring only a material balance of water flow into and out of the system.

The water balance models have been used extensively in predicting leachate quantity and aiding design of landfills. Water balance model predictions may be suspect due to the questionable accuracy of the input parameters, such as, rainfall, evapotranspiration, permeability, and refuse moisture storage estimates (Bagchi 1994; El-Fadel et al. 1997). The second approach to predicting landfill flow is using finite-difference/finite-element solution techniques. Many investigators have taken this more complex approach of using the unsaturated flow theory through porous media to predict landfill leachate flow (Korfiatis 1984; Straub and Lynch 1982a, 1982b). The Hydrologic Evaluation of Landfill Performance (HELP) is the most widely used model which employs the same concept of water balance excluding the biochemical reactions and computes the leachate quantity based on a detailed meteorological and waste characteristics data.

The chemical composition of leachate depends on many factors such as the initial composition of solid wastes, particle size, degree of compaction, hydrology of the site and the age of the tip, the last one reflecting the degree of stabilization of the wastes. The quality of leachate is further affected by several pretreatment methods or management practices such as the shredding of wastes, the separation of recoverable material (i.e. paper, aluminum, and glass), the recycling of leachate back to the landfill, or the co-disposal of municipal wastewater sludge. For these reasons great variations in the quality of leachates produced from different landfills have been reported in the literature (Johansen and Carlson 1976; Chian 1977; Bookter and Ham 1982; Harmsen 1983; Kouzeli-Katsiri et al. 1993; Robinson 1993; Gettinby et al. 1996; Farquhar1989).

The concentration profiles of several leachate constituents measured at the bottom of laboratory-scale columns filled with compacted municipal refuse have been reported in the literature by several authors (Pohland 1975; Fungaroli and Steiner 1979; Raveh and Avnimelech 1979; Ham and Booker 1982; Ehrig 1988, 1991; Colin 1990). Although wide differences in leachate composition were noted, in general the concentration curves resemble a hydrograph with a rising limb, a peak, and a recession limb.

Attempts to model and predict municipal solid waste leachate composition and volume have generally followed two approaches. The first and simplest approach has been to fit empirical equations to contaminant concentration curves (Raveh and Avnimeleh 1979; Wing 1979; Reitzel et al. 1992). The contaminant curve is generally developed as either contamination concentration versus time or cumulative leachate volume per unit mass of refuse. Although these models capture the general decrease from high initial concentrations to low final concentrations, they tend to be site-specific, to the particular landfill or lysimeter.

In the second approach, models were developed that quantitatively describe the biological processes occurring during leaching. The model used almost exclusively by several authors was developed originally by Straub and Lynch (Straub and Lynch 1982a). These authors used four process equations to describe the solubilization of organic matter, the degradation of soluble organic matter and the growth of acidogenic and methanogenic anaerobic biomass. A pseudo first-order kinetic was used for the solubilization rate whereas the Monod model (Monod 1949) was used for the degradation rate and microbial growth rates. The lysimeter was simulated as a series of fully mixed reactors. They used data from the literature to calibrate the models and derived values for the kinetic coefficients. A satisfactory simulation was obtained when the reactor was depicted with four fully mixed compartments in series. In a second paper, Straub and Lynch (Straub and Lynch 1982b) used the theory of unsaturated flow through porous media to describe the flow through the lysimeter. The flow equations were solved explicitly. Using the same kinetic parameters calculated previously, these authors found no significant differences between the two models. They concluded that the fully mixed reactor assumption offered a good simplification for the simulation of the decomposition processes in lysimeter. The same unsaturated flow model was used by Korfiatis and Demetracopoulos (1984) and Demetracopoulos et al. (1986). These authors employed a different numerical scheme to solve the flow equations and performed a sensitivity analysis for the evaluation of equation parameters. Their results were not, however, verified with the experimental data. The simple four-reactor model developed by Straub and Lynch (1982a) was subsequently used by Colin (1990) to predict the leachate quality from laboratory lysimeters. Gonullu (1994a, 1994b) followed the same model but with a single fully mixed reactor to predict the quality of leachate from two laboratory-scale lysimeters. Using multiple correlation analysis of the experimental data, he developed two equations to predict the maximum concentration of substrate in the effluent and the value of the microbial kinetic coefficient.

Kouzeli-Katsiri et al. (1993, 1999) developed a simple mathematical model to simulate solid waste decomposition in landfills. They took into account the processes of exchange of the organic matter between the solid and the liquid phase and the depletion of the dissolved organic matter due to biological decomposition and flushing. They employed two equations using first-order kinetics to describe these processes. In their study the landfill was considered as a single fully mixed reactor. Their model has a smaller set of parameters than other models proposed in the literature. They calibrated their model using the results obtained from six pilot laboratory-scale lysimeters. They proposed kinetic parameters for characteristic types of solid wastes and landfill management practices. They concluded that due to the very slow rate of reactions, the landfill can be considered as a single fully mixed reactor with a volume equal to the volume of the water content of the deposited mass of refuse at field capacity. They described complex processes occurring during refuse decomposition by two first-order reaction equations for organic matter solubilization and soluble organic matter depletion. The total leachable mass of COD was expressed as a fixed percentage of the total mass of wastes. Their model thus contained three kinetic parameters, the values of which were obtained from the experimental results.

The most commonly reported danger to the human health from these landfills is from the use of groundwater that has been contaminated by leachate. Leachate may contain dissolved or suspended material associated with wastes disposed off in the landfill, as well as many byproducts of chemical and biochemical reactions. Leachate tends to migrate in surrounding soil and may ultimately result in contamination of underlying soil and groundwater. Landfill leachate has the potential to contaminate the surrounding environment and impair groundwater use. To protect the groundwater from contamination by landfill leachate, it is quite essential to provide the bottom barrier of suitable thickness, and to minimize the amount of water that could enter the landfill to create leachate.

The design of an earthen barrier system in the case of buried waste traditionally is based on the assumption that the hydraulic conductivity controls the rate of leachate migration. However, recent field studies have indicated that the diffusion is the controlling mechanism of contaminant transport in fine grained soil (Bagchi, 1994)

Rowe (1988) discussed in detail the mechanisms that control the migration of contaminants and the method of modeling the same. He also discussed ways to determine diffusion and distribution coefficients and introduced the concept of equivalent leachate height. These concepts have been used in formulating the model used in this study.

Conca and Wright (1990) determined diffusion coefficients of unsaturated gravel and showed that the quartzite gravel had the lowest coefficients. Likewise Rowe and Badv (1996a, 1996b) determined the dispersion and diffusion coefficients of chloride in clayey silt, silt and sand for Darcy velocity of the order of 0.018 m/yr. In their study, the effective diffusion coefficient in saturated fine gravel was found to be about 93% greater than that for unsaturated fine gravel. Tests conducted in low velocities encountered in a typical landfill site and the modeling results show that there is no significant mechanical dispersion evident in clay, silt or sand under such low flow rate.

Shakelford and Daniel (1991) determined the factors that affect the diffusion in aqueous or free solutions and suggested that diffusive transport is slower in free solution as compared to soils. Later, Shakelford and Redmond (1995) also obtained the hydrodynamic dispersion coefficient in fine grained barrier material and concluded that diffusion dominates miscible transport at low flow rates in such barriers. Rowe et al. (1988) obtained diffusion and distribution coefficients in saturated undisturbed clayey soils and validated their theoretical model using data obtained from an experimental study. They determined the diffusion and distribution coefficients of a contaminant using undisturbed clayey soil samples while maintaining a Darcy velocity in the range of 0.025 to 0.035 m/yr. This range exceeds the normal velocity in most practical applications involving clayey liners. According to them, the contribution of mechanical dispersion to the hydrodynamic dispersion coefficient at a velocity of 0.035 m/yr or less is negligible for clay and diffusion is the dominant mechanism for contaminant transport through clay for seepage velocity between 0.064 m/yr to 0.09 m/yr.

Rowe and Booker (1995) analyzed one dimensional contaminant transport model for single solute in a layer of finite thickness. In their study, they considered the combination of the effects of advection, diffusion, dispersion and chemical retardation due to finite quantity of pollutant in the landfill overlying a clay deposit with moving groundwater beneath the clay deposit. Their model also included leachate mounding and the effect of clogging of leachate collection system due to the contaminant. However, their analytic solution was suitable only for cases where homogenous liner thickness was small compared to the dimensions of landfill.

Rowe and Booker (1995) applied semi-analytic technique to a barrier system consisting of a primary leachate collection system, a primary clay liner, a secondary leachate collection system and secondary liner system overlying a fractured aquitard located above an aquifer. This new technique accommodates the varying properties of the system, complex source concentration history and movement of contaminant through a fractured aquitard. The time and cost involved while performing the analysis is also relatively less, however it's application is limited to a horizontally layered soil with the soil properties being same at ant horizontal location within the layer.

Similarly some researchers have developed the models (Benson and Daniel 1994a) that predict the minimum thickness of the soil liners using a stochastic approach but they only considered advective flow in saturated soil. Further Benson and Daniel (1994b) discuss some difficulties encountered in the analysis of soil liners. They suggested that the performance of soil liners depend on the minimum travel time and the magnitude of flux subsequent to the first passage measured using probability theory to incorporate the spatial variations of the hydraulic conductivity properties. They recommended minimum thickness of compacted liners should be 0.6m to 0.9m with respect to spatial variability of hydraulic conductivity.

Jessberger (1997) discussed controlled landfill design and highlighted the various aspects of design followed in European countries. Rowe and Nadarajah (1993, 1997) presented analytical methods of predicting the velocity of low beneath the landfill and ways of evaluating the hydraulic conductivity of aquitard.

Munro et al. (1997) investigated the retardation of contaminants in shallow clayey soils and later validated their field studies with analytical models of Sudicky (1988). Sawbrick (1994) gave a comprehensive list of all the processes that take place in waste deposits.

Boateng and Cawlfield (1999) conducted a two dimensional sensitivity analysis of contaminant transport in the saturated zone and suggested that the bulk density and distribution coefficients are significant for reactive transport.

An analytical model to assess the risk of groundwater pollution caused by leaks from solid waste depositories was developed by Rudakov and Rudakov (1999). These analytical modeling approaches for design of clayey liners are simple to use, but overly conservative, hence one needs numerical solution.

Yan et al. (2006) presented a contaminant transport model based on fuzzy set theory to simulate water flow and contaminant transport in the unsaturated soil zone under surface ponding condition. They found that saturated hydraulic conductivity is the most sensitive to model outputs among all the parameters. They used trapezoidal fuzzy numbers to express the uncertainties associated with saturated hydraulic conductivity.

Tsau and Aarne (1998) discussed improved landfill liner for the prevention of groundwater contamination due to heavy metals. They found that the hydraulic conductivity of clay lime, first increases with increasing lime and thereafter decreases with increasing lime. They concluded that the incorporation of lime in landfill liners reduces hydraulic conductivity, increases puncture resistance, and enhances heavy metal capture capacity.

Mitchell and Madsen (1987) reviewed literature on test data related to chemical effects on clay hydraulic conductivity and found that in general inorganic chemicals may influence hydraulic conductivity of clays through their effects on soil fabric. The results indicated that the reduced electrolyte concentration led to a substantial decrease in hydraulic conductivity because of dispersion.

Benson and Daniel (1990) suggested that to achieve low hydraulic conductivity, it is necessary to destroy clods either by wetting the soil at high moisture content or using large compacting effort. Gracia-Bengochea et al. (1979) concluded that hydraulic conductivity is controlled by changes in macropore distribution rather than porosity. Mitchell et al. (1965) studied the effects of moisture content on hydraulic conductivity and concluded that higher moisture content result in dispersed structure, resulting in lower value of *k*.

Liner amendments are incorporated into landfill liner materials to impart special properties to it, for the purpose of retarding the migration of specific contaminants from landfill leachate. In this study liner amendments have been tried for the retardation of organic contaminants from landfill leachate. These amendments are supposed to sorb the organic contaminants and thus impede the movements of such contaminants to the bottom of landfill liner.

Generally compacted soil liner in the thickness of 1 to 1.2m is considered adequate to minimize the rate of contaminant migration by reducing the advective transport of contaminants through the liner. However, even with hydraulic conductivities less than 10^{-7} cm/s, the migration of many organic contaminants through compacted clay or composite liners by simple Fickian diffusion can be significant (Shakelford 1991; Park and Nibras 1993; Gullick 1998).

The magnitude of organic contaminant sorption to natural soils and clays is related to the solubility of organic contaminants and the organic carbon content of the sorbent (Chiou 1998; Bartelt Hunt et al. 2003).

The replacement of inorganic cations such as sodium in clay minerals by quaternary ammonium cations through the cation exchange reaction reduces the hydration of the clay and decreases its surface negative charge. Such replacement results in change in properties of the clay change significantly from being highly hydrophilic to increasingly organophilic. The sorption characteristics of clays amended by quaternary ammonium compounds, called organoclays to ionic and non-ionic organic compounds are mainly due to the partitioning process or hydrophobic interactions (Lo et al. 1998; Budhu et al. 1997). Organoclays effectively minimized the impact of phenolic wastes on cement hydration (Lo and Liljestrand 1996). The study on organoclays as waste containment barriers for pollutant attenuation have demonstrated it to be a attractive alternative to the conventional clay liners [Xu et al. 1997; Lo et al. 1997]. Increasing the sorption capacity of the earthen liner effectively reduces the contaminant advection velocity and the rate of solute diffusion during transient solute transport without requiring an increase in liner thickness (Lo 2003; Gullick and Weber 2001). Although a sorbed contaminant will eventually desorb from the liner, the mass flux from the bottom of the liner is significantly reduced when an organobentonite is a component of the liner (Bartelt Hunt et al. 2003).

Shanon et al. (2005) studied the sorptive capacities of granular activated carbon (GAC), shale, benzyltriethylammonium-bentonite (BTEA-bentonite), and hexadecyltrimethylammonium-bentonite (HDTMA-bentonite) for organic compounds. They carried out the experimental studies for sorption of organic compounds by these four materials to evaluate their effectiveness for use in compacted clay landfill liners. They carried out laboratory sorption and permeability experiments on these materials with 0.002 N $CaSO_4$ solution (to simulate the ionic strength of ground water) and a synthetic leachate. From the results of their experimental investigations with 0.002 N CaSO4, they concluded that all four amendments have a very high sorptive capacity for the three organic solutes viz. benzene, trichloroethylene, and 1,2-dichlorobenzene. They found that GAC exhibited the highest sorptive capacity for all three solutes, followed by BTEA-bentonite, HDTMA-bentonite, and shale. They carried out permeability tests on specimens composed of Ottawa sand, untreated bentonite, and either 3 or 9% amendment by weight. Their Results indicated that all amended specimens had a hydraulic conductivity less than or equal to 1x 10^{-7} cm/ s, with the exception of the specimen amended with 3% GAC, which had a measured hydraulic conductivity value of $2x10^{-7}$ cm/s. They found that the changing of permeant fluid to a synthetic leachate had little effect on the overall hydraulic conductivity of the specimens. They further carried out one-dimensional benzene transport simulations for the complete evaluation of these materials for their incorporation into a compacted clay liner. They found that all four amendments can effectively retard the transport of benzene through the liner.

Tarek et al. (2002) conducted field studies to determine if the field hydraulic conductivity of barrier layers constructed with foundry green sand is comparable to the hydraulic conductivity measured in the laboratory on laboratory-compacted specimens. Their field hydraulic conductivity was measured using sealed double ring infiltrometers, two-stage borehole permeameters, and on large block specimens. They found that the field hydraulic conductivity data followed the same trends with bentonite content and liquid limit observed in the laboratory. Testing after winter exposure showed that the field hydraulic conductivity was unaffected by winter weather, even though the test pads underwent up to six freeze-thaw cycles. They concluded that, exposing the test pads to summer weather had no measurable effect on the field hydraulic conductivity. The results of their field study validated that foundry sand as a useful industrial by-product that can be beneficially used as a hydraulic barrier material.

Lo (2003) studied the feasibility of using modified clays in conjunction with conventional soil-bentonite admixtures as a waste containment barrier with the objective of determining the abilities of such admixtures to act as a sealant and retain dissolved organic contaminants. They prepared five kinds of soil admixture for the batch sorption, compaction, and hydraulic conductivity tests. The first three kinds of soil admixtures consisted of 80% natural soil with 20% organoclay; 10% bentonite and 10% organoclay; 20% bentonite. The fourth soil admixtures consisted of 70% natural soil, 20% bentonite, and 10% organoclay. The last admixture was composed of 60% CDV, 20% bentonite, and 20% organoclay. The various percentages of each soil type were determined on a dry weight basis. The natural soil used in their study was completely decomposed volcanic rock (CDV), yellowish brown in color. Modified clay used in their study was obtained as manufacture product from Biomin International, USA. They carried out experimental investigations on these admixtures for the determination of sorption capacities and hydraulic conductivity. The sorption isotherms of total organic carbon, by five types of soil admixtures were nonlinear. They concluded that the soil specimen with more modified clays exhibits higher organic sorption capacity and a larger retardation factor. The specimens with 20% of pure bentonite by dry weight have a higher optimum water content and plasticity. They found that with the addition of bentonite in the soil material consisting of natural soils and modified clays, the hydraulic conductivity to leachate decreases from about 1 x10-^{7} to 1x 10^{-8} cm/s. No incompatibility between landfill leachate and soil specimens was found. They applied curve fitting technique to determine the effective porosity and hydrodynamic dispersions of both Chloride and Total Organic Carbon. They carried out 1D transport model simulations to develop design charts for the determination of liner thickness using such admixtures. They concluded that that the presence of modified clays in the soil containing 20% bentonite largely reduces the required liner thickness.

Loretta and Franky (2001) used Bentonite, forest soil, and spruce bark and subjected these materials to batch adsorption testing, leaching cell testing, and selective sequential extractions (SSEs) to investigate the heavy-metal compatibility of clay barriers and the potential of forest soil and spruce bark as clay barrier materials. They concluded that the materials can be ranked on the basis of sorption capacity as forest soil > bentonite = spruce bark. The hydraulic conductivity values of heavy metal leachate were two orders of magnitude greater than those of the blank (0.01 mol calcium nitrate) leachate. The forest soil admixture ranked first in terms of heavy-metal retention capacity and breakthrough points.

Jaruwong and Wibulswas (2003) studied the effect of Quaternary Ammonium Cations (QACs) carbon chain length used for modifying Montmorillonite clay to examine its adsorption properties towards humic acid utilizing a batch equilibration technique. They performed a comparative using Montmorillonite modified with 3 different QACs, which include Tetramethylammonium (TMA), Hexadecyltrimethylammonium (HDTMA) and Benzyldimethylhexadecylammonium (BDHDMA). The Batch adsorption results demonstrated that HDTMA-Montmorillonite and BDHDMA-Montmorillonite are better adsorbent than the precursor and TMA-Montmorillonite. The adsorption affinity of Montmorillonite towards humic acid increased with an increase of the QACs Carbon atoms in the organo-clays. Solution of TMA, HDTMA and BDHDMA were used separately as the modifying agent in the preparation of organoclays. The synthesis was carried out in a batch reactor with 0.5 litres of each QAC solution. The adsorption isotherms were conducted using a batch equilibration technique in a 0.5 litre conical flask by varying the initial humic acid concentration while maintaining the amount of adsorbent clay. A control sample with no adsorbent was also prepared. In addition, comparative adsorption experiments were conducted with the untreated Montmorillonite as a reference material. The flasks were closed and shaken overnight to allow equilibrium to be achieved.

Tsau and Aarne (1998) studied the impact of addition of lime to a mixture of montmorillonite clay, and sand. The results of laboratory experiments conducted by them involving test of hydraulic conductivity against Zinc Chloride solution, suggest of changes in chemical & physical properties of clay because of lime addition. They found that the hydraulic conductivity of lime clay mixture increases with increase in percentage of lime to an extent and decreases thereafter with further lime addition. They concluded that the lime addition to clay results in increase in hydraulic conductivity with increase in lime content upto about 5-10%. With lime content greater than this hydraulic conductivity decreases. Higher amount of lime in clay results in greater amount of Zinc capture delaying Zinc breakthrough.

6.0 Organization of thesis

Chapter I Deals with issues of concern in landfilling of waste particularly groundwater contamination due to migrating leachate from its bottom. This chapter presents the objectives and motivation of study. The scope of study is also delineated.

Chapter II presents the review of the published literature on the various issues concerning landfilling of waste.

Chapter III discusses the mechanisms involved in migration of contaminants from landfill into the clayey barrier/ aquitard. A mathematical model representing migration of contaminants from landfill facility is formulated taking into account the various contaminant transport mechanisms. The solution of the model is implemented in Matlab using finite difference method subjected to suitable initial and boundary conditions. The developed is validated for two cases of field data from published literature. Sensitivity analysis of the model has been carried out. Advantages and limitations of the model are been discussed.

Chapter IV deals with application of the model developed in the present study for a actual landfill site located at Delhi. Groundwater samples in the vicinity of landfill and in the direction of groundwater flow have been collected and analysed to arrive at the possibility of groundwater contamination due to landfill. The results of simulation are in conformity with actual results of groundwater sampling and analysis in the immediate vicinity of landfill. The possible impact of landfill leachate on the groundwater of region in future has been analysed. Gas production from the landfill has been estimated using a public domain software based on first order kinetics.

Chapter V presents design charts for the minimum thickness of liner. The design charts have been obtained by simulation of the model for range of Darcy velocities. The results of simulation have been plotted as maximum relative concentration with respect to depth for a suitable design period. The design charts are presented for both conservative and non conservative contaminant and for a range of waste density represented by equivalent leachate height.

Chapter VI deals with the experimental studies carried out various admixture obtained by adding bentonite, organoclay and natural soil in varying proportions for the determination of sorption and hydraulic characteristics of such admixtures. Different Admixtures have been prepared using synthesized organoclay and manufactured organoclay. Experimental results of such admixtures have been used in Transport Modeling Simulations to determine their effectiveness with regard retardation of organic contaminants and consequent impact on thickness of liner. The admixture with higher content of organoclays have indicated higher retardation capacity of such liners with respect to organic contaminants and consequent reduction in the minimum thickness of liner required.

Chapter VII **CONCLUSIONS** presents the conclusions, recommendations from the present study and future scope of the work in the field. One dimensional contaminant transport model has been developed and implemented to simulate the transport of contaminant from a landfill facility to an underlying aquifer through a saturated clayey barrier. The model developed in this study has been validated successfully for two cases of field data. Sensitivity analysis of the model has been performed which has demonstrated the diffusion dominating mechanism for the case of contaminant transport from landfill into clayey barrier. Model developed in the present study has been applied on Bhalaswa landfill site at Delhi. Design charts have been prepared for the determination of minimum thickness of landfill liner for a range of permeability values and equivalent height of leachate for conservative contaminant in landfill leachate. The design charts have also been prepared for a range of permeability values and retardation factors for non-conservative contaminant. These charts can be used for the determination of minimum barrier thickness on the basis of maximum concentration of contaminant species of interest and pollution prevention criteria. Experimental studies were carried out on eight admixtures (five prepared using synthesized organoclay and three prepared using manufactured organoclay with bentonite and natural soil as remaining constituents) for the determination of their hydraulic and sorption characteristics. The results of such studies have been used for transport modeling simulations to determine the effectiveness of each of these admixtures with regard to retardation of organic contaminants from landfill leachate and consequent impact on minimum thickness of liner based on organic contaminants. The results of transport modeling simulations can be used as design charts for the determination of minimum liner thickness based on organic contaminant, and using these admixtures. Retardation factor of the admixtures was found to increase with increase of organoclay content. The sorption behavior of all the admixtures have been found to be consistent and scalable.

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CONTAMINANT TRANSPORT FROM LANDFILLS

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By
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CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Study was carried out focused on the mass transport of contaminants from landfill leachate, and possible control measures for the same. Following conclusions were drawn from the present study:

1. The landfills of the National Capital Territory (NCT), Delhi, cumulatively generate a significant amount of leachates, alarming in terms of its groundwater contamination potential. Viewed from the international standard India is water-stressed today and is likely to face severe water scarcity by 2050. Delhi, as the rapidly growing capital city of Asia, is facing problems in terms of both the groundwater quality and quantity. Disposal of solid waste in landfills is a cause of concern because of its groundwater contamination potential, due to migration of landfill leachate.

2. The severity of the groundwater contamination is significant on account of associated complexity in pollution source identification, limited feasible options for treatment of groundwater all being cost prohibitive, complicated process in fixing geophysical boundaries, difficulty in prediction of movement, insufficient dilution, slow movement, lack of any natural cleansing capability. As a sizeable population in India is dependent on groundwater as the only source particularly densely populated, urgent attention to activities of solid waste disposal by landfilling is highly needed.

3. Engineered landfills constructed with suitable components based on sound engineering principles can substantially reduce the environmental risks posed by the same. Liners provided at the bottom prevent migration of leachate and consequent contamination of groundwater. Landfill covers provided at the top substantially reduce the amount of water percolating the landfill and consequent generation of leachate. Leachate is collected through primary and secondary collection system and disposed off after adequate treatment. Landfill gas is collected through gas collection layer and is sent for energy recovery/flaring.

4. Effective pollution control through the proper design of landfill and leachate management facilities requires an understanding of leachate quality and quantity. Leachate quantity is usually modeled and/or determined using a simple water balance approach taking into account the amounts of water entering the landfill (i.e. precipitation, waste moisture in excess of moisture holding capacity of the waste and additional water input such as water in wastewater treatment plant sludges if allowed) and the amounts of water leaving the landfill (i.e. water consumed in biochemical reactions and evaporation). The chemical composition of leachate depends on many factors such as the initial composition of solid wastes, the particle size, the degree of compaction, the hydrology of the site, the climate, and finally the age of the tip reflecting the degree of stabilization of the wastes.

5. Migration of landfill leachate to the bottom and sides, results in contamination of groundwater and soil. The impact of landfill leachate on the contamination of groundwater depends upon various parameters viz. properties of flow and media through which flow of leachate takes place, and the properties of contaminant, density of waste etc.

6. Landfill gas is the product of a series of complex reactions involved in the decomposition of organic matter that produce methane and other gases and compounds. Methane is a valuable energy resource and also a potentially dangerous gas that must be controlled and regulated

7. One dimensional contaminant transport model has been developed and implemented to simulate the transport of contaminant from a landfill facility to an underlying aquifer through a saturated clayey barrier. The model developed in this study has been validated successfully for two cases of field data. Sensitivity analysis of the model has been performed which has demonstrated the diffusion dominating mechanism for the case of contaminant transport from landfill into clayey barrier.

8. Solid waste generation in Delhi has been found to be about 6000 metric tonnes per day. The per capita waste generation stands at about 500 gm per person per day, about five times of national average of per capita waste generation. Solid waste management in Delhi is mainly based on the disposal of the waste through the three operational landfills located at Bhalaswa, Okhla, and Gazipur. All of these sites are being operated on the principle of open dumping of waste. None of these landfills is being operated as a sanitary landfill.

9. Bhalaswa landfill is located in one of the most urbanized north west corner of Delhi, occupying area of about 21.06 hectares of land, and receives about 300 tonnes of municipal solid waste per day. The geology of the area around Bhalaswa landfill is mainly alluvium. The landfill at present receives about 2500 tonnes per day of waste out of which about 700 tonnes per day goes to

compost plant. The landfill is publicly owned and is managed by MCD. The incoming wastes originate mainly from households and commercial areas, but there are also some wastes that are brought to the landfill from industries.

10. Model developed in the present study has been applied on Bhalaswa landfill site at Delhi. Simulated Chloride concentration at a depth of 10 m below landfill was found to be consistent with the observed Chloride concentration in groundwater at the same depth. As per the results of simulation studies Chloride concentration at the same depth is expected to multiply by about four fold after next 15 years.

11. The concentration of all the parameters in the groundwater samples collected from the vicinity of Bhalaswa landfill site has been found to be decreasing in radially outward direction from the landfill.

12. Characterization studies of landfill leachate form Bhalaswa landfill show a high concentration of Chloride, TDS and BOD. The presence of iron to the extent of 8.2 mg/L along with chromium, lead, zinc, copper indicate the dumping of hazardous waste from nearby industrial areas involving in pickling work.

13. The total gas emissions from Bhalaswa landfill were found to be 2400 million cu.m. for methane and carbon dioxide. Maximum emissions rates for carbon dioxide and methane were found to be about 250 cu.m. per sq.m. per year.

14. Design charts have been prepared for the determination of minimum thickness of landfill liner for a range of permeability values and equivalent height of leachate for conservative contaminant in landfill leachate. The design charts have also been prepared for a range of permeability values and retardation factors for non-conservative contaminant. These charts can be used for the determination of minimum barrier thickness on the basis of maximum concentration of contaminant species of interest and pollution prevention criteria

15. Experimental studies were carried out on eight admixtures (five prepared using synthesized organoclay and three prepared using manufactured organoclay with bentonite and natural soil as remaining constituents) for the determination of their hydraulic and sorption characteristics. The results of such studies have been used for transport modeling simulations to determine the effectiveness of each of these admixtures with regard to retardation of organic contaminants from landfill leachate and consequent impact on minimum thickness of liner based on organic contaminants. The results of transport modeling simulations can be used as design charts for the determination of minimum liner thickness based on organic contaminant, and using these admixtures. Retardation factor of the admixtures was found to increase with increase of organoclay content. The sorption behavior of all the admixtures have been found to be consistent and scalable.

16. Use of organoclays in landfill liners is an attractive option for the sorption of organic contaminants as the same shall reduce the minimum thickness of landfill liner where a high amount of organic contaminants in landfill leachate are observed.

7.2 Recommendations

On the basis of the present study, following recommendations are made:

1. Solid waste disposal practice being followed in Delhi and in all other municipalities in India mostly consists of open dumping of wastes without any regard to observance of sound engineering principles for the disposal of solid wastes. Any new landfill should be designed and constructed on the basis of sound engineering principles.

2. A landfill should not be constructed without adequate provision of bottom barrier for the prevention of groundwater contamination due to migrating leachate. The minimum thickness of bottom barrier may be determined by referring to the design charts prepared in this study. A bottom barrier is highly essential in suitable thickness for the protection of groundwater sources in the country which are already under pressure from contamination by a large number of sources. Providing a thickness of liner beyond than that required to meet the pollution prevention criteria of the region would be wasteful of land resource.

3. Bhalaswa landfill was found to be contributing to the groundwater contamination in its vicinity. The landfill is being operated as open dumping facility. The leachate from the landfill needs to be collected, isolated and disposed off in a proper manner after suitable treatment. Migration of landfill leachate should be prevented for the protection of groundwater in the area.

4. Methane gas generation from the Bhalaswa landfill has been found to be considerable in terms of quantity. Landfills should be provided with collection and utilization system for the methane gas either for recovery or for flaring to

avoid any possible hazard. The energy recovery will not only provide revenue from the landfill but will also reduce emission of greenhouse gases and the same can be traded as carbon credits.

5. For the purpose of construction of bottom barrier, locally available materials may be used if these satisfy the design of permeability. If locally available materials are not suitable for providing minimum permeability, then bentonite is can be added to reduce the permeability of liner material.

6. Wherever the landfill leachate contains a high concentration of organic contaminants, a landfill liner should be provided amended by the addition of organoclay. Design charts prepared in this may be used for the determination of liner thickness to be provided in such a case.

7. Wherever, landfill leachate contains inorganic or conservative contaminants in the concentrations greater than that of organic or non conservative contaminants, it is recommended to provide the bottom barrier amended by organoclay, and a secondary leachate collection system, for the collection treatment and final disposal of landfill leachate containing, high concentration of inorganic contaminants.

7.3 Scope of Future Work

The research work in any area can never be said to have achieved its completion, and the same applies to the present study as well. The work carried out in this study can be extended as under:

1. The model developed in this study can extended to take into account the effect of consolidation of clayey barriers on contaminant transport from the landfills.

2. Studies on liner materials can be extended for the sorption and retardation of various other contaminants in landfill leachate.

3. Vulnerability of a landfill site to groundwater pollution can be assessed using the tools of remote sensing and GIS.